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Engineering News-Record

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April 26, 1928

Earthquake-Resistant Construction

By Henry D. Dewell—The first of two articles outlining what is known today of the problem of building to resist earthquakes

405-Foot Arch Dam for Owyhee Project

By J. L. Savage—Details of approved design for unusually high arch-gravity type dam for U. S. Reclamation Service in Oregon

Also

A Graphic Method of Calculating Flood Storage

Locating Railway From Mexico City to Pacific Coast

Foreshore Protection Work Along Chesapeake Bay

Concrete Paving - Base Control by Core Tests

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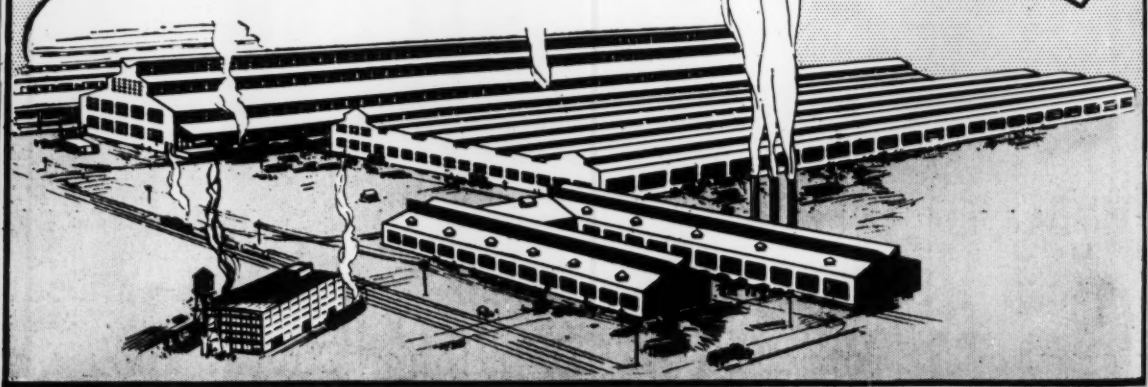

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Business Plus

"WE WELCOME you to our family," so begins a letter from an electric light company to a new customer. The letter carries a constructive message of co-operation. Soberly considered, it is unnecessary, since it adds nothing to the definiteness of the business agreement set up between the two parties. But by undertaking to write co-operation and mutual obligation into the actual if not into the legal contract it contributes that profoundly important element which converts mere business into business plus. That element is one without which the interrelations of men must always fail of their fullest fruition, and business needs it ever more vitally as the complexities of industrial life increase. Not only public utilities but brick makers, steel fabricators and concrete mixer manufacturers have a place for it. It may not produce better mixers, but who will deny that it can smooth the path of progress toward the ultimate ideal of perfect concrete!

Secretive Administration

TIME was when public affairs were the affairs of the public, and officials were its representatives and servants. We are far along in political development and have nursed up many little autocracies. The attempted suppression of the Mississippi River Commission's report on flood control last fall was an amusing instance, so far as such things can be amusing. Another, of more recent date, is the concealment by the State Department of the Special International Niagara Board's progress report on improvement of Niagara Falls. That report, though made in the last analysis to the public and by an agency of the public, not of the State Department, has been withheld from other than brief outline summary publication. Apparently the other party to the negotiation, Canada, does not regard the matter as confidential in the diplomatic sense, for English technical journals published the details of the proposed control works a week or two ago. Few modern things have had so intricate and so interesting a history in technical discussion as conservative control of Niagara Falls, and the problem in essence is wholly an engineering one, with diplomacy in only second or third position. Under the circumstances, the withholding of a public report appears in the light of another instance of unfortunate practice of secretive administration.

Nearing a Vote

TO SPEAK with hope rather than confident expectation, a vote on the Mississippi flood-control bill seems to be drawing near. At the moment a tumultuous, not to say chaotic, condition prevails in the halls of Congress and along the private-messenger routes that connect the Capitol with the White House. The situation is replete with personal, political and local demands. The Valley representatives, with a small number of exceptions, insist on loading the whole burden of cost on the nation at large. Their logic is not clear, for they say that the land to be protected cannot pay for the cost, while yet asserting that the land is worth protecting; but there they stand. Urgings such as repeatedly advanced in these columns, that an economic study be made before deciding on what work is worth doing and where the benefits lie, appear to have found no understanding. On the other hand, the President is said to hold out for two main points: Local payments for levee rights-of-way and control of the work by the army engineers; yet the question of floodway damages cannot be disposed of by merely ignoring it, and the virtue of army-engineer control still awaits demonstration. Thus at present it is not possible to chronicle any constructive developments in this badly tangled and yet vast and momentous undertaking. While the situation contains no engineering elements at the moment, one of the prime elements of trouble in the legislative controversy seems to be due to the existence of a ready-made engineering plan, regardless of the fact that the crux of the problem is the uncertainty and unexplored character of many basic engineering factors. Doubtless the army engineers who were led into producing the plan under pressure from the administration have since wished that they had insisted upon only a rough outline of a plan and no estimate of its cost, since more could not be done until ample time had been provided for a study of all the facts.

The Unquiet Earth

A SERIES of destructive earthquakes in the Balkan region during the past ten days draws attention again to the instability of the earth's crust. There are those who claim that the world just now is an epoch of specially active seismic disturbance. But since long-time records in the matter are scarce and imperfect, it is safer to regard the present earthquake frequency as normal. In any event we are bound to think of the security of our great works of construction with respect to earth movements. The subject is of broad concern, and ultimately, through the involvement of financial structures and general welfare, it touches every man. Knowledge of the subject is still vague and uncertain, however. Two years ago *Engineering News-Record* presented a general review of earthquake disturbance as it affects the engineer's needs. In the present issue the work then begun is carried forward by H. D. Dewell's article on earthquake-resistant building construction. It expresses more clearly than has been done by others what is known and what remains indeterminate as to the ability of high buildings to live through earthquakes. It does not give assurance that we can defy the earth's shakings, but this is hardly surprising, and matters of vastly simpler kind also permit of only qualified answer. The engineer who tries to equip his bridge with a railing strong enough to restrain wild

vehicles is likely to find that some vehicles and conditions of wildness can break through the strongest railing he would build. So also in earthquake resistance: We need not expect to attain absolute security from damage, but we can readily provide some measure of resistance. Mr. Dewell's statement of the case tends to lead to more general appreciation of the importance of providing earthquake strength in buildings—and of the possibility of doing so at moderate cost.

Bonuses and Water-Works

OFFERING bonuses to induce industries to locate in a community has long been held unsound, yet many smaller communities continue to do it. Wherever the results of this practice have been investigated, according to the Chamber of Commerce of the United States, more failures than successes were found. Between two and three million dollars was poured out by a mid-West city in bonuses to industries, by donation of free sites, by tax exemption or by selling of stock; today the industries are not in existence. Free water or water below cost is one of the worst forms of bonus, because it places an unfair burden on the water-works. Often the added consumption required by the industry requires enlargement of the works. The additional expenditures do not produce a corresponding increase in revenues, or else produce no increase at all. Where industries require large amounts of water, engineering studies may show that the development of an independent supply would be advantageous to both the industry and the community. As a rule the cost of water represents but a small percentage of the total production cost in any industry. If it can be shown that free water is essential, the industry is more likely to be a liability than an asset to a community. In short, all bonuses at municipal cost, whether in the form of free water, free sites or remission of taxes, are an injustice to other industries and to the taxpayers, and in the long run are a damage to a community. They have been under general disapproval for years past. That disapproval should become universal.

Progress Through Difficulty

HUMAN achievement advances by dint of overcoming difficulties. As the day's work confronts us with harder problems, we strive to improve our methods and develop our art. By this course civil engineering in particular has made its significant advances, in the time of the lake dwellers and the temple builders and even to this day. The account of how the great Owyhee River dam was designed again emphasizes this broad truth. The magnitude of the problem and its heavy responsibilities were the motives for exhaustive studies of strength and security. Analysis was carried far beyond the extent customary in the engineering of dams. An unusual type of structure proved to give most efficient utilization of materials—an arch dam of almost full gravity section, yet rigid enough to throw the larger part of the resistance into the horizontal arches. As it will be the highest dam yet built by man, both dimensions and type were compelling reasons for searching study of strength. That the foundation called for scrupulous, thorough investigation is perhaps obvious in view of the failure of the St. Francis dam; but the resulting discovery of a wide brecciated fault and the decision to carry a cutoff down more than 100 ft. below the dam base in order to seal it are none the less striking. Altogether the Owyhee dam advances the frontiers of the art. It takes rank with the Coolidge dam in originality and distinction, but, since it is less far removed from normal types and requirements, it is likely to exert an even greater influence on technical practice.

Changes in Water-Works Ownership

NEXT Tuesday the voters of San Francisco will for the second time within less than a year and for the fifth time in less than twenty years cast their ballots for or against buying the property of the Spring Valley Water Company. The city is doubly committed to municipal ownership of water-works: First, by a charter declaration for ownership of all utilities, and second, through having been engaged for years in the execution of its Hetch Hetchy dam and aqueduct project, of which the dam and part of the aqueduct is completed, as well as a power plant that is producing a large yearly revenue. Besides these commitments, San Francisco feels the urge of the country-wide movement, now more than a century old, for municipal ownership of water-works.

At the close of 1800 all but one of the seventeen water-works in existence were privately owned. Since then practically all of the cities and towns of 1800 have changed to municipal ownership, as also have hundreds of cities in which, subsequently, water-works were privately built. Today, of the 68 cities of 100,000 population in 1920, only eight have their works still in private ownership. Of the eight, San Francisco is the largest. Oakland, one of the eight, is also committed to municipal ownership, through having joined with other cities across the bay from San Francisco in the Mokelumne gravity water-supply project, now well on toward completion.

In the East, Paterson is in the same general state as San Francisco and Oakland, with prospects of early municipal ownership. The New Haven authorities have just decided not to exercise a purchase option. A near-by city, Bridgeport, seems equally content with the service it receives from a private company, as also does Indianapolis. The private works at Scranton have just gone into the hands of one of the large holding companies and have been consolidated with the private works at Wilkes-Barre and between that city and Scranton. The eighth privately owned plant in the 68 cities, at Birmingham, is also owned by a large holding company.

Altogether it may be expected that of the eight cities mentioned four if not five will continue for some time to have their water supplied by private companies. In other words, among the 68 cities of 100,000 population or over in 1920, municipal ownership, now so nearly complete, promises to stop short at the five cities enumerated. This may mean ending for a time, when well-nigh complete, a remarkable change from private to municipal ownership. In cities of less than 100,000 population there has been a strikingly similar change, less sweeping for the whole country, but almost complete in a number of states.

What effect the extensive corporate acquisition of privately owned plants in our smaller cities will have on the strong trend to municipal ownership is an interesting subject for speculation. In so far as the change results in improved service made possible by more competent business and engineering administration and more adequate capital, holding-company ownership should lessen if not entirely remove the demand for public ownership arising from inadequate service. Yet that part of the demand for

a change which is based on objections to absentee ownership will probably increase, no matter how good the service, and regardless of the fact that in many communities absentee ownership is not new.

Where the works bought by holding companies are physically consolidated with others in the same territory, of which the most notable case thus far is afforded by the Scranton and Wilkes-Barre consolidations, municipal ownership will come less readily than acquisition of works by individual cities. The greatest obstacle which advocates of municipal ownership will have to surmount in the case of plants that go into possession of holding companies may arise from overcapitalization, unless this is held in check by state utility commissions. High purchase prices paid to local companies, followed by bond issues to cover those prices and capital stock issues on top of the bonds, would stand in the way of municipal or district purchase, since once such things are permitted it is hard to keep overcapitalization from entering into purchase prices. On the other hand, overcapitalization and its use for rate structures would also stimulate a demand for public ownership.

Thus far few large private companies have been acquired by holding companies, and very little physical consolidation has resulted or can result from the holding-company movement to date. Almost all the large water-works are publicly owned, as has already been shown. Moreover, occasional talk to the contrary notwithstanding, there has been scarcely an instance, as yet, of a holding company buying out a municipally owned water-works.

On the whole, if the movement toward municipal ownership of water-works slows up from now on, it is more likely to be due to the fact that most of the plants, especially the larger ones, are already owned by the cities they supply than to the activities of holding companies.

Early Findings Confirmed

UNIFORMLY, the several investigating boards which studied the St. Francis dam disaster have reached the conclusion that it was a pure foundation failure. Thus they confirm fully and precisely the preliminary findings of Mr. Bowers' examination of the dam immediately after its failure, reported by him in our March 22 issue. The latest verdict is that rendered by the board of which Dr. Elwood Mead was chairman, briefly summarized in the news pages of this week. The surveys and tests made by this board contribute much valuable information. Most vital is the determination, by tests, that the "rock" in the west abutment of the dam was so unconsolidated as to go to pieces within a few hours when placed in water. This condition, together with the fissured nature of the ground and the presence of a bad geological fault crossing the foundation, showed clearly that the disaster was inevitable, waiting only for the complete filling of the reservoir and a short period of percolation and saturation to work out its evil course.

Disaster was wrought into the very project of the dam; yet there were able engineers engaged in the work, and they had done brilliant things in the past. Nothing has been said in any of the findings that would explain or reasonably account for the choice of site of dam and method of preparing the foundation. Possibly this omission is due in part to the difference of view among the engineers who have examined the site. This difference means only that the chosen site was so questionable as to demand most anxious scrutiny of geological and physical factors—such scrutiny as is common in the case of large dam projects, but in this one case omitted or slighted. Whatever the magnitude of the human error responsible, a fatal fault lay in the administrative system that permitted the hazardous conception and defective execution to come to pass.

Frost Damage to Highways

FROST damage is a large annual charge against highway upkeep. No state in the Northern frost zone escapes its yearly bill for repairing "boils" and "heaves." Traffic every spring is denied the full use of highways until the roadbeds have settled and the maintenance crews have restored stable conditions of foundation and surfacing. It is impossible to estimate the annual cost of this frost action. As the preliminary records of the past winter, in this issue and that of a week ago, clearly indicate, each season presents a different accounting. But in the round of years the accounting runs into huge sums, both in repair costs and in charges for delayed traffic. These sums represent a tax on insufficiency in highway construction and maintenance. They are being too much regarded as payment for an act of God.

The reports on frost damage in this and the preceding issue are presented merely as a target for thought. They furnish almost no quantitative information; they offer only surmises of fundamental causes. They do, however, signify the broad facts that frost damage is general and of large dimensions; that it is not escaped by even the highest types of road structure and increases as the type declines in quality; and that weather, the relative succession of rain, of frost and thaw, and of snowfall, is a highly influencing factor. In the background cheer or gloom is evidenced over this year's behavior of roads, and here and there is apparent a combative disposition not to accept Nature's vagaries as supreme facts. To summarize a prevailing mental condition: Frost damages are great, uncertain and providential, and are accepted as they come, with an occasional act of rebellion.

Perhaps the thought impresses one too strongly that highway officials have supinely accepted the hazards of frost. At least no criticism can be made of their alertness and energy in repairing damages and in curtailing traffic to preserve frost-weakened thoroughfares. They have organized their forces of restoration to an admirable perfection. Have they been as alert and energetic in organizing for immunity and prevention? If they have not, there is now evidence that they are approaching that situation. Here and there a student is looking into the phenomena of frost action and seeking to draw from his researches some rules that the practical roadbuilder can apply. A notable example is the work of V. R. Burton in Michigan, as described in our issue of last week. Coming issues will present other examples and discussions. Outstanding is the fact that this spring the Bureau of Public Roads has its investigators working in several states with state highway department engineers.

It is important that a change in mental attitude is being developed. The "spring break-up" has too long remained unprotested as an evil both to the roadbuilder and to the traffic which the roadbuilder serves. It is time that its claim to an annual tribute from every community which has roads and a cold winter should be contested. There is promise in the recent activities of highway engineers that the contest has begun.

Earthquake-Resistant Construction
I—Data of Design

Divergent Opinions of Engineers—The Nature of Earthquakes, Their Periods, Amplitudes and Accelerations—A Design Value of Intensity—Long and Short Columns—Natural Periods of Vibration of Buildings

By Henry D. Dewell
Consulting Engineer, San Francisco

Two Years Ago the author discussed "Earthquake Expectancy in California" for readers of *Engineering News-Record* (Jan. 21, 1926, p. 114) and pointed out the importance in the design of buildings and engineering works of providing strength against earth disturbances. The present series of two articles is the sequel to the former article. It outlines what is known today of the problem of earthquake-resistant building, a problem that is yet far from being fully solved. The first article, on the underlying facts and data, will be followed by one on design principles and methods. —Editor.

MUCH difference of opinion exists with respect to what constitutes proper design for resistance to earthquakes. Reference to various articles that have appeared from time to time, particularly since the Santa Barbara earthquake of 1925, in technical magazines, trade bulletins, special reports of various building material manufacturers and other publications will indicate that while all agree that competent and honest design and workmanship are essential for satisfactory performance of a building during an earthquake, the necessity and feasibility of definite earthquake-resistant design is controversial.

The special committee of the American Society of Civil Engineers which investigated and reported on the damage of the 1906 California earthquake ventured the opinion that buildings designed to resist a wind pressure of 30 lb. per sq.ft. of exposed area would safely withstand an earthquake of the intensity of that of 1906. This latter statement has been widely quoted and rather widely accepted, without investigation of the evidence on which it is based or the limitations which the authors implied. Consequently it has been applied to conditions for which it was not intended.

Flexible or Rigid Construction—Some engineers argue that the earthquake-resistant structural frame should be flexible; they say that the framed structure with diagonals must surely have those diagonals ruptured in a heavy shock. Others believe that the structure should be as rigid as possible against shock. Still others advance the opinion that extensive bracing of a tall building is, to a great extent, ineffective; that rigidity cannot be attained except at prohibitive expense, and that a tall building will best resist an earthquake if its first-story columns are made long and flexible. And so we have the condition existing today that engineers interested in building construction are divided in opinion regarding proper construction against earthquakes.

The failure of engineers to agree on what constitutes proper knowledge of earthquake-resistant construction is not surprising. Only in recent years have engineers in America given serious thought to the subject. Earthquake phenomena were studied in the past by the geologist and physicist, and in comparatively recent years by the student of seismology and the geophysicist. The structural engineer has been slow in taking up the problem. Without an underlying knowledge of seismology he has been at a disadvantage, and the press of active work has hindered him from studying intensively what has appeared to be not only a difficult subject but one so complex that it has at times seemed hopeless to look for a practical solution. The relatively long periods between destructive shocks in any one locality on this continent have also had their effect in retarding continued intensive study of the problem.

The engineer is primarily interested in the effect of that class of earthquakes known as "tectonic," as distinguished from those caused by volcanoes. The tectonic earthquake is believed to be caused by a slipping along a fault plane of one block of the earth's crust upon another, this slipping and resultant elastic rebound sending out elastic waves throughout the materials of which the earth is composed. It is a definite fact that elastic waves in the earth do occur at the time of earthquakes, for they are registered by seismographs wherever such instruments are set up.

Nature of Earthquakes

These earthquake waves are complex in nature. In general, two principal types are to be distinguished—longitudinal and transverse. The longitudinal waves are similar in character to sound waves, in that the vibrations are in the direction of propagation and are those of compression and dilatation. The transverse waves, on the other hand, are akin to light waves, in that the vibrations are transverse to the direction of propagation. The effect of passing through or impinging upon media of varying densities in the earth's crust is to cause refraction and reflection. These phenomena, together with the interference of the waves and other matters not so well understood, result in the complexity of the waves noted in the record of the seismograph. The first earthquake waves to be recorded at any point of observation are longitudinal waves; later, the transverse waves are received. Thus it has been a relatively common experience in earthquakes for observers to note that the shock came first from one direction and then from a second, the two directions being frequently at right angles to each other.

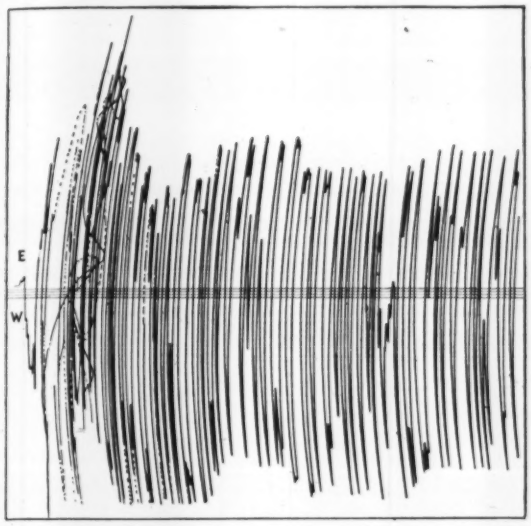
Dr. Perry Byerly, seismologist at the University of California, is authority for the following statement:

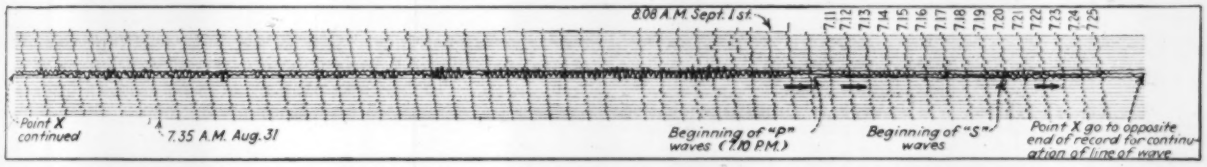
> As recorded on the seismograph, three groups of waves are to be distinguished. The first to arrive are composed largely of the compressional, rarefactional or longitudinal type; they have traveled through the earth by a curved path with a dip deeper than a chord. Their velocity is about 5.6 km. per second. The waves of the second group are of the shear or transverse type; and have traveled a similar path to the waves of the first group, but with a lesser speed, about 3.2 km. per second. These first two groups are termed the first and second preliminaries.
>
> The third group of waves appear on the seismogram after the

second preliminaries and usually carry the maximum energy at distant stations. They have traveled along the earth's surface and are therefore referred to as surface waves. Within these surface waves are at least two separate types, and possibly more. The motion in the first type seems to be traverse to the path and largely in the horizontal plane. The velocity is about 4.4 km. per second. The second kind of surface waves has a vertical component, and they have a velocity of about 3.9 km. per second. On some records the so-called "maximum portion" arrives as something definitely new with a velocity around 3.3 km. per second. Often this portion builds up gradually and appears as a part of the previous type. The maximum portion has a larger vertical component, and the waves resemble, or are, surface waves.

Following the maximum portion the record dies out in a long series of oscillations, which are often referred to as the coda.

In nearly all earthquakes the second preliminaries and the maximum portion or principal portion are usually inseparable if not identical. . . .





FIGS. 1 AND 2—PART OF MAXIMUM PORTION OF THE TOKYO EARTHQUAKE OF SEPT. 1, 1923

Fig. 1, above—From the record at the Tokyo observatory; original magnification 2. Fig. 2, below—From the record at Berkeley, Calif.; original magnification 48.

Many observers have noted waves in the alluvium at the time of a severe earthquake. These are obviously a secondary effect. Their velocity is quite small relative to those of the waves above described. Observers relate seeing them approach and bracing themselves for the movement. These waves may be very effective in damaging buildings on alluvium. These waves are not recorded on seismographs, since (1) whenever possible seismographic stations are located on rock, (2) such extreme motion would disable ordinary seismographs built for delicate work.

Longitudinal Waves

In a few instances, where the focus of the earthquake was near the observer, the first effect felt has been a sudden "shove" or "bump" followed by a rapid swinging motion. This was the testimony of observers at Santa Barbara on June 29, 1925. Students of seismology are not in agreement on the importance to engineering structures of the effect of the first longitudinal wave. Dr. Bailey Willis, past-president of the Seismological Society of America, gathered evidence in Santa Barbara which led him to believe and state that the collapse of certain structures was coincident with the arrival of this first shock, which struck the foundation of the building as with a blow. Seismographic records do not show the importance of the first longitudinal wave from the standpoint just mentioned. However, it must be remembered that seismographic records are generally, if not always, recorded on instruments at points safely remote from the focus of the earthquake. Thus it may be that a structure within the seismic zone must be capable of resisting the stresses set up by two conditions of forces: First, that of a sudden impulsive blow; and second, the series of rapid oscillations that follow the first shock. The large shaking table, recently constructed at Stanford University to experiment on the effect of vibrations in structures, provides for a combination of an impulsive blow and subsequent vibrations.

Fig. 1 shows a part of the "maximum portion" of the seismogram of the great Japanese earthquake of Sept. 1, 1923, registered at Tokyo, while Fig. 2 shows a similar part of the seismogram of the same shock as registered at Berkeley, Calif. In comparing the two seismograms, the reader must bear in mind that the Tokyo record has a magnification factor of 2, while the Berkeley record has a static magnification of 48. Such magnification applies only to the amplitudes. The periods, as measured to the proper scale, are correct as shown, except as the natural periods of the two instruments tend to emphasize similar periods.

Periods, Amplitudes and Resonance

The period of the "maximum portion" of a destructive earthquake has been generally taken at from 1 to 1½ seconds, based on observations by Omori and others. Prof. A. Imamura states that in the Japanese earthquake of Sept. 1, 1923, the principal vibration registered had a period of 1.35 seconds and a double amplitude of 8.86 cm. (3.46 in.) and that this was followed by a vibration having a period of 2 to 3 seconds and a double amplitude of 20 cm. (7.8 in.). Omori estimated that in San Francisco, in 1906, on made ground, the period was about 1 second and the amplitude 2 in.

If the natural period of a structure should synchronize with that of an earthquake having a constant period, the destructive effects would be increased. Collapse of the structure would be not only a possibility but a probability, depending on the amplitude and duration of the earthquake vibrations.

The crashing of hanging lamp shades against the ceilings of office buildings in Tokyo in the earthquake of 1923 indicates that resonance for at least a few vibrations may occur in an earthquake.

Seismograph records such as are reproduced in Figs. 1 and 2 show that an earthquake wave is not one of simple harmonic motion. This means that the periods, amplitudes and accelerations vary throughout the time of duration. Dr. Perry Byerly has plotted the periods

and amplitudes of the record of the 1927 Calexico earthquake, as recorded at Pasadena. This graph shows that both periods and amplitude are constantly changing. The period varies from 0.20 to 3.2 seconds. One period of 0.07 second was constant for 5 seconds.

Fig. 3 shows the variation in the periods of the long swings in a part of the "principal portion" of the east-west component of the Tokyo earthquake, as taken from the seismogram Fig. 1. The periods are seen to range from 0.80 to 5 seconds, although the longer swings are broken up by very short undulations with periods of approximately 0.70 second (not shown in the chart). This study indicates that long-continued resonance between the structure and the earthquake wave, so feared by many writers, is not probable, but that any structure whose natural period of elastic vibration is in excess of 1 second, when subjected to an earthquake of major intensity, may suffer the effects of resonance for a few vibrations.

Accelerations

Actual accelerations have been measured in many earthquakes by seismographs. Unfortunately, such instruments are usually located on rock or good soil and therefore do not record the maximum accelerations developed, since these maxima occur in loose or unstable ground. Furthermore, seismographs are not located in the areas where the greatest concentration of building values exists. Occasionally, as in the case of the San Francisco earthquake, even the nearest seismographs were so situated and constructed as to be incapable of recording the waves of maximum amplitude. Accelerations as published are often estimated from the damage done to structures in particular localities as compared with the damage done to similar structures in the neighborhood of a seismograph. The overturning of small structures, such as stone monuments, by the earthquake has been utilized to compute the accelerations necessary for overthrow. The latter method of estimation is likely to result in errors of considerable amounts.

Accelerations ranging from 0.003g to 1.00g have been observed or estimated in earthquakes. An intensity of 0.4g, or more than 12 ft./sec.^{2}, is not uncommon in great shocks; but this, as well as the highest intensity just noted, is of such local occurrence that from the standpoint of general structural design it need not be considered. In the San Francisco earthquake in 1906, the estimated acceleration of the shock at Third and Market Sts., 7 miles from the fault of origin, ranged from 0.06 to 2.6 ft./sec.^{2}, or from 0.02 to 0.08g. In this same shock the estimates of acceleration in made and marsh land ranged as high as 10 to 12 ft./sec.^{2}. An acceleration of from 0.09 to 0.10 of gravity, or 3 ft./sec.^{2}, has been advocated for design, on the basis of studies of the damage done by the Japanese earthquake of 1923. The writer understands that the Tokyo building regulations require the use of an acceleration of one-tenth of gravity, with the maximum unit stresses for a combination of dead loads and earthquake loads not to exceed the usual working unit stresses. It should be noted, however, that much larger accelerations were made evident in that shock, and also that the source of the shock was nearly 60 miles distant from Tokyo.

Intensities of Earthquakes

Unquestionably the Japanese earthquake of 1923 was of far greater intensity than the 1906 earthquake in San Francisco. The relative accelerations of the shock in the two business districts was roughly 3 to 1. Indeed, the San Francisco earthquake was more nearly of the intensity of the Japanese earthquakes of February, 1922, which Dr. Naito, of Waseda University, described as moderately severe. This apparent fact should not be lost sight of in estimating the seismic risk of structures. San Francisco has been accused, and justly so, of attempting to minimize the severity of the 1906 earthquake. Nevertheless a study of the records of damage done to buildings will indicate that the accelerations in the principal business district, as estimated by Ried and Wood, are reasonably accurate. The writer has recently examined several photographs of the business district streets taken after the earthquake and before the fire had reached these streets. In these photographs the absence of débris in the streets is noticeable, and even parapet walls, for the most part, are seen to be intact. Testimony of engineers who went to their offices in the morning of the earthquake bears out this statement of minor damage to the substantially constructed buildings. It should be emphasized that these facts indicate the rather moderate severity of the shock rather than the strength of the structures.

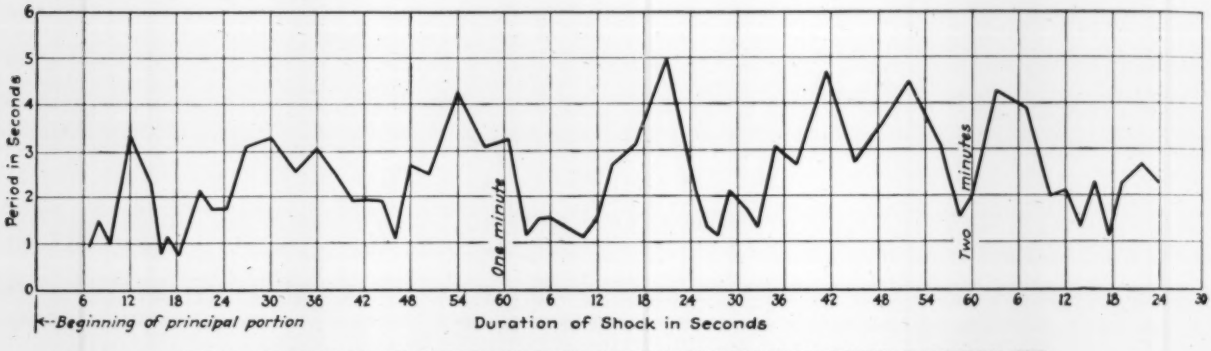


FIG. 3—VARIATION OF PERIOD OF EAST-WEST COMPONENT IN TOKYO EARTHQUAKE

The Charleston earthquake of 1886, the San Francisco earthquake of 1906, the Tokyo earthquake of 1923 and the Santa Barbara earthquake of 1925 may all be rated as having reached Grade X (the highest) in the Rossi-Forel scale, in certain limited areas; yet it is probable that no American city has ever experienced so severe a shock as have the cities of Japan.

If historical records are to be followed, it would seem that for American conditions, in the case of buildings on firm soil at a distance of several miles from the nearest

fault plane likely to be active, the value of one-tenth of gravity, or 3.2 ft./sec.^{2}, as a design acceleration is a reasonable maximum. For a combination of dead and reasonable live load stresses, with the stresses due to earthquake, an increase of unit stresses of 50 per cent would seem conservative, and reasonable assurance of safety would be had if this maximum fell well within the elastic limits of the materials used.

In the immediate vicinity of the fault plane the acceleration will be much higher, while any building situated immediately over a fault plane on which differential movement occurs will probably suffer rupture, irrespective of the strength of the structure.

Estimating Earthquake Forces

The power of an earthquake to injure structures is commonly measured in terms of the horizontal component of acceleration of the elastic wave. The vertical component is generally neglected, as it is almost always smaller than the horizontal. A body weighing W lb. (and therefore having a mass of W/g) and moving with an acceleration of a feet per second requires a force of $F = Wa/g$. This relation permits a calculation of the force which an earthquake exerts upon a building, substantially anchored to its foundation, as soon as the acceleration of the earthquake wave is known, provided the structure is assumed to be rigid and all parts are simultaneously given the acceleration of the ground.

The elastic wave of the earthquake is assumed to be of simple harmonic nature. If the period and amplitude are known, as from a seismographic record, the maximum acceleration may be computed from the formula

$$Acceleration = (4\,\pi^2 \times Amplitude) \div Period^2$$

When seismic records are not available, the acceleration must be estimated. By virtue of the assumption set forth in the preceding paragraph, the shear and bending moment on any horizontal section of a vertical structure results from a force acting at the center of gravity of that part of the structure above the section considered. The force is taken as the product of the mass of the structure above the section by the assumed acceleration. The shear is equal to the force. The moment is the force times the distance of its line of application above the section.

Emphasis must be placed on the fact that this method of calculation ignores the possibility of elasticity in the structure and rocking of the structure on its base. For the case of buildings of limited height, or with a limiting ratio of breadth to height, properly designed to resist such earthquake acceleration and to give the maximum rigidity possible, experience indicates that they will suffer only trival, if any damage, from a major shock.

Design Against Earthquakes

Two methods of design of buildings for resistance to earthquake shock are available. The first is to design the structure and its foundations in such a way that the energy of the shock will not be transmitted from the foundation to the building, or, rather, so that only a small residual shock will be transmitted. Buildings so constructed may be described as having free foundations. The second method is to connect the building firmly to its foundations and then design the structure so that it may safely resist the stresses set up by the full accelerating force of the earthquake wave; such buildings may be said to have rigid foundations.

Free Foundations—In the past, some small buildings and some lighthouses in Japan have been provided with free foundations, by rollers placed between foundations and superstructure. Seismograph records taken during small earthquakes have shown that the effect of the roller foundation has been to reduce the swaying of the building to a negligible amount. This construction has, however, not yet been worked out in practical form for general application to large buildings.

The possibility of simulating the free foundation method of design by utilizing certain characteristics of earthquake waves is worth noting. The surface waves of an earthquake are the most destructive, and certain experiments have given evidence that the intensity decreases within a few feet in depth of ground. Practical application of this phenomenon does not seem impossible. Certain types of buildings, such as buildings of public assembly, located well within the property lines and having basements under the entire floor, might without undue expense be surrounded with an areaway so that the building would have no contact with the ground surface except through joints designed to allow movement in the case of earthquake. In some commercial buildings it would seem practicable to accomplish a similar result by carrying all walls, including the first-story wall, on the structural frame and avoiding their contact with the surface of the ground; likewise the basement columns might be kept free from contact with all retaining walls.

Various modifications of the free foundation method of design, in which artifically located points of contraflexure combined with slip joints of limited movement are located at the bottoms or at the tops of the first-story columns, have been proposed, but none of these proposals has had practical application. Mention has already been made of the fact that some engineers believe that long slender first-story columns will approximate the effect of actual hinges. This type of design will be described later in some detail.

Rigid Foundations—In the usual construction, superstructure and foundations are tied together. Therefore we are for the most part concerned with the type that has previously been termed rigid foundation. Here it is obvious that the building must be sufficiently strong to resist, as a vertical beam, the shear and bending moment resulting from the earthquake shock and the inertia of the building.

Omori's Short and Long Columns

Literature on the design of structures against earthquake contains many references to the observations of Fusakichi Omori, the late Japanese seismologist, on the action of brick factory chimneys in Japan in earthquakes and to his experiments on models on a shaking table. Indeed, Omori's shaking table experiments are classic in earthquake literature.

Omori's work has so influenced all subsequent investigators and writers that a short discussion of his beliefs seems necessary. He divided all structures into two classes, which he designated as "short columns" and "long columns." His basis of classification was the ratio of the natural period of the structure to that of the earthquake. If this ratio was less than unity or but slightly in excess of unity, the structure was classed as a "short column," and the greatest destructive effect of the earthquake was held to be expected in the lower portion of the structure. On the other hand, if the ratio was in excess of unity,

the structure was classed as a "long column," and he would expect the destructive effects of the earthquake to be evidenced in the portions at some height above the base. In particular, Omori stated that his observations showed that tall free-standing factory chimneys broke not at the base but at a point about two-thirds of the height. In accordance with Omori's observations and writings, the term "center of percussion" has come to be associated with tall, slender structures.

The term "center of percussion" has to do with the action of a sudden or impulsive force acting upon a rigid body free to move. A tall, slender, rigid body of uniform moment of inertia, free to move, acted upon by an impulsive force at its base, will tend to rotate about an instantaneous center of rotation located at a point two-thirds of the height above the base. The resultant shear and bending moments for this case have been treated by Prof. Joseph N. LeConte in the January (1927) *Proceedings* of the American Society of Civil Engineers, and discussed in subsequent issues, to which the interested reader is referred. Omori did not, however, as far as the writer knows, compute the distribution of shears and bending moments in such chimneys under the conditions of instantaneous rotation.

American engineers who have had occasion to investigate the seismic stability of buildings, chimneys and other structures have in general adopted Omori's conclusions, although the same methods for determining the stresses in the structures of both classes have generally been employed. When the structure has been assumed to possess a "center of percussion," the effect of such action has usually been taken care of by employing a higher effective acceleration for determining the stresses in the portion of the structure near the assumed center of percussion than has been used in computing the strength of the sections near the base.

A study of the individual cases of the list of broken chimneys which Omori cites indicates that while the average position at point of fracture was about two-thirds of the height, the variation from this average was great. The writer's studies of this subject lead him to the tentative conclusion that the distribution of moments and shears for tall chimneys is approximately in accordance with the curves shown by Professor LeConte in the paper to which previous reference has been made.

Action of Elastic Structures

No structures are rigid; all possess some elasticity, and some are to be definitely classed as fully elastic rather than rigid structures. The effect of elasticity is an important factor in the nature and amount of the stresses set up in a structure by an earthquake.

An actual office building with its rather complicated structural frame and its wall coverings pierced by openings is too complex a structure to be given a rigid mathematical treatment when its action under the influence of an oscillating force is investigated, even though such an oscillating force is assumed to be of simple harmonic nature. This fact has led many to conclude that a building cannot be designed to resist the forced vibrations of an earthquake. However, if the investigation be confined to the case of an elastic vertical beam of uniform moment of inertia under the action of an oscillating force of simple harmonic type, the resultant distribution and amount of shears and bending moments may be found. From such studies much valuable information may be gained on the action of the actual building.

The maximum stresses set up in such an elastic beam of uniform moment of inertia by an oscillating force of simple harmonic type are proportional to those found by considering the beam to be perfectly rigid and acted upon by a constant force at the base. Such stresses may be greater or smaller than those resulting from the assumption of perfect rigidity; the determining factor is the ratio between the natural period of free vibration of the beam and the period of the oscillating force applied to the beam. In other words, these stresses are independent of the dimensions, weight and elastic modulus of the beam, except as such factors affect the natural period of vibration.

For a beam fixed at the base the maximum moments and shears occur at the base. If the beam is hinged at the base the case is different. This condition has not yet been fully worked out. It may be said at this time, however, that for a certain ratio of the natural period of the structure to that of the simple harmonic wave the stress conditions are the same as for the case of a rigid beam acted upon at the base by an impulsive force.

A certain length of time must elapse for these conditions to materialize. The above conditions are, therefore, to be regarded as limiting. While the results are not exactly applicable to a building, they indicate the nature and amount of probable stresses in buildings.

Natural Period of Vibration

The exact value of the natural period of vibration of a building is difficult to compute, because of the complexity of its structure. It is possible, however, to determine the approximate value of the period for many buildings, and such approximate value will be sufficient for practical purposes. The building may be considered as a vertical beam of uniform moment of inertia fixed at its base.

The actual period of elastic vibration of such a beam is

$$T = 1.785 \frac{(W')^{\frac{1}{2}} h\, h^{\frac{1}{2}}}{(EA)^{\frac{1}{2}} r\, g^{\frac{1}{2}}}$$

in which E is the modulus of elasticity, W' the total weight and A the cross-sectional area of the beam, h is its height, r the radius of gyration of its cross-section about the axis of bending, g is the acceleration of gravity, and T is the period of free oscillation.

If the dimensions of a building, its breadth, length, height, thickness and material of walls, and the size and spacing of its columns be known, an approximate value for the natural period may be calculated. As an indication of what the actual values of the period of buildings are, reference is made to an article "Vibration of Buildings" by Prof. Elmer E. Hall, *Engineering News*, Aug. 1, 1912, p. 198. The following values are extracted from the article referred to. By means of a seismograph Professor Hall measured the natural periods of elastic vibration of a number of buildings in San Francisco.

	Sec.
Three-story brick building, 50 ft. wide	0.53
Six-story brick building, 48x130 ft.	0.70
Eight-story reinforced-concrete building, 92x128 ft.	0.40
Steel-frame office building, stone and brick walls, 75 ft. square and 315 ft. high	2.32
Twelve-story class A office building, brick walls	1.37

Subsequent measurements on actual buildings indicate that a modern 30-story office building, with the shortest base dimension about 0.35 of the height of building, with structural steel frame, with reinforced-concrete floors and brick curtain walls, and designed for ordinary wind loads, will have a period of natural vibration of about $2\frac{1}{2}$ seconds.

These experiments confirm what would be expected—viz., that the period of vibration as measured at any point in the height of the building is a constant, but that the amplitude of vibration is proportional to the height. The amplitude depends upon the disturbing force; hence only the relative amplitudes at various heights in any one building are of interest.

Omori states that the natural period of vibration of a building is a measure of its rigidity. In the case of certain buildings in Japan the effectiveness of bracing added after completion of the building has been checked by means of measured periods of vibration taken before and after the work of reinforcing the building.

Foreshore Protection Along Chesapeake Bay

By Lloyd T. Emory
Consulting Engineer, Philadelphia, Pa.

FORESHORE protection is becoming increasingly imperative not only where property is exposed to erosion or damage from the open sea but also in more secluded waters. This article deals with the latter character of work.

Before describing in detail the wall which is the subject of this article, there are three general observations the writer would like to make: First, the foreshore should be built up and held by the proper use of groins; second, if a wall is used, it should be placed above or at the normal high-tide line; and third, vertical faces of any large extent should never be used to oppose wave action.

FIG. 1—COFFERDAM IN PLACE
Driving sheet piling at the toe of the slope with the aid of a water jet.

There are few places on Chesapeake Bay and its tributaries where a rock foreshore exists, therefore erosion is more or less general except where sand, assisted by vegetation, is able to take care of the situation. Normally the sweep of the tide would not be strong enough to constitute a destructive force, but nevertheless the tide is an assisting factor. In the area of the bay under consideration there are a number of grasses, shrubs, trees and vines that thrive either wholly or partly in the brackish waters. Before the country was settled and the shore line cleared, it is probable that the beaches were stable and little erosion took place. With the clearing of the country this condition has changed and with the building of expensive summer homes near the water's edge beach protection becomes of increasing importance. It was such a situation that the writer was called upon to consider in providing shore protection at Pioneer Point Farms, Queen Anne's County, Maryland, for the estate of Mrs. Helena S. Raskob.

FIG. 2—FORMS IN PLACE FOR CASTING ALTERNATE PANELS
Note the boards are added as the concrete is brought up the slope.

Some protective measures had already been taken on this estate. A vertical board bulkhead 1,800 ft. long had been constructed and three long loose rock groins placed at irregular intervals. The foreshore was scouring badly and the timber bulkhead had already been undermined at two points for a considerable distance. The first recommendation was to cut off the long rock groins and use the material removed therefrom for a system of short groins placed at regular intervals.

The bank above the wooden bulkhead had been graded to a 1 to 2 slope and sodded at considerable expense. In order to disturb it as little as possible, construction plant had therefore to be placed with an eye to the conservation of this sodded slope.

Information preceding the design was obtained on the average and maximum tides and on the location of a local benchmark from the director of the U. S. Coast and Geodetic Survey. Offshore depths of water were taken from the coast chart and supplemented by sounding in the immediate vicinity of the foreshore. Current directions were studied during several tides by floats.

There are four points of interest in the design, which provides essentially a paved slope cast in 12½-ft. slabs, reinforced by galvanized metal mesh and rods: (1) Concrete was carried down to 1½ ft. below low water and the toe protected by 3-in. tongue-and-grooved sheetpiling driven down 3 ft. or more into the stiff clay underlying

FIG. 3—COMPLETED WALL, SHOWING READJUSTMENT OF THE BEACH SAND AFTER REMOVAL OF COFFERDAM
The sand ultimately should lie against the top step.

the beach sands; (2) the lower portion of the slope is stepped to retard wave action, and it has been found by experience that the beach sands will conform more readily and remain in place better where such a surface is used; (3) the top of the slope is protected by an overhanging curved coping, for the purpose of stopping any wash that may run to the top of the slope; and (4) the joints between the panels are stiffened by precast beams or slabs. Besides stiffening the face, these slabs also help to prevent the joint calking from being driven through the joint.

Drainage was provided for the fill back of the slope by a line of open-joint farm tile at the elevation of normal high tide. Every 200 ft. this drain was connected with

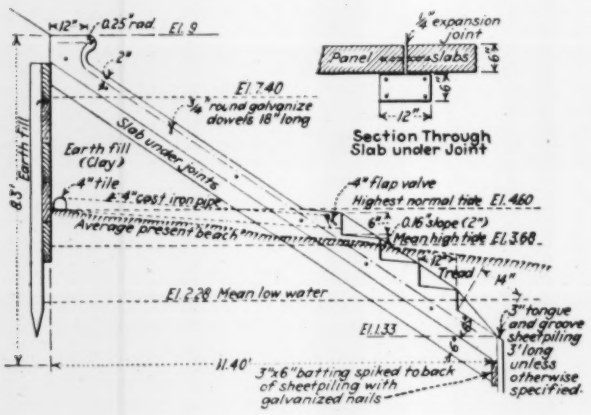


FIG. 4—DETAILS OF CHESAPEAKE BAY SEA WALL

weep holes in the face of the wall. These openings were protected by cast-iron flap valves with bronze seats. Except for these openings, every effort was made to make the face of the wall as tight as the bottom of a ship. The panels were poured in alternate sections and premolded expansion-joint material was used to seal the joint, being placed before pouring the intermediate slabs.

In order to minimize the destructive force of sea water aided by such mechanical forces as abrasion and frost, a 1:2:4 mix was used and every effort made by tamping and finishing to have the concrete as impervious as possible. Particular attention in pouring the concrete was given that critical area which lies between normal high and low tide. It is also hoped that the rock groins will stabilize the beach sands and so build up the foreshore that the sand will cover this critical area and the wall will come into actual use only during storm periods and excessively high tides.

Field, Barker & Underwood, engineering contractors, of Philadelphia, executed the work under the general direction of their president, Howard Underwood.

24,000,000 Pesos for Chilean Road Construction in 1928

The extraordinary budget for 1928, which was recently passed by the Chilean Congress, provides for 24,000,000 pesos (the Chilean peso equals $0.122 at present rate of exchange) for road construction in Chile during the coming year. Of this amount 5,000,000 pesos is for completion of the road from Valparaiso to Casa Blanca; 15,000,000 for construction of roads already determined, and 4,000,000 for construction of the San Bernardo highway.

Job Charges for Plant Rental Maintain City's Equipment

FOR several years past in the engineering department of Flint, Mich., no funds have been asked of the budget for the maintenance of the material yards nor for the purchase of new equipment. A revolving fund is kept intact by charging each job with proportionate rental and depreciation. The engineering department handles all of the construction work, in addition to the planning, supervision and maintenance, with a large trained personnel and well-stocked yard of equipment for all regular municipal undertakings. Contract work, according to H. C. McClure, city engineer, is recommended only when the structures are of a special, nonrecurring nature for which the city does not have the equipment and organization. Included in this special list are buildings, bridges, large special sewers and such large items as the units of the sewage-disposal plant. In consequence of this action an equipment fund has been set up and the attempt is made to operate and finance the yard just as it would necessarily have to be handled by a contractor.

As equipment is repaired, or new equipment purchased, money is taken from this equipment fund, and as the equipment so purchased is used, the rental charges are credited again to this fund. In this manner, the equipment fund is self-sustaining, as it must be in every contractor's organization and is in every industrial business that plans to keep in business for any length of time. In this way the job on which equipment is used pays for that equipment only in proportion to the use made of it, and the general taxes are not increased through budgets by large requests from year to year for new equipment.

However, Mr. McClure states that the records of production and cost on the several items of construction work would not have been possible were it not for the fact that many men in the organization who are in direct charge of the construction activities have been with the city for a number of years and include some of its most loyal, long-time employees. The personnel of the department generally, and the actual outside working force, he feels, have the interest of the city at heart to an extent as great as or greater than would ordinarily be found in the general contractor's organization.

High-Pressure Locomotive Tested

Toward the end of 1927 the Swiss Locomotive & Machine Works of Winterthur built a trial high-pressure steam locomotive after several years of research work, preparatory trials and stationary tests, according to *The Engineer*, London. The boiler of this locomotive works at a pressure of 350 lb. per sq.in., and a high-speed, three-cylinder, non-compound engine operates the driver through a geared jack shaft. In the middle of January, 1928, comparative road tests were made with a conventional superheated steam locomotive of 170 lb. per sq.in. pressure and of equal output, in order to ascertain the savings of coal and water obtainable with the high-pressure locomotive. Non-stop runs were made on consecutive days, the composition of the train and the weather being the same for each two comparative runs. The results showed that a saving of coal of from 35 to 40 per cent with respect to the horsepower-hours at the drawbar and a water economy of from 47 to 55 per cent were obtained.

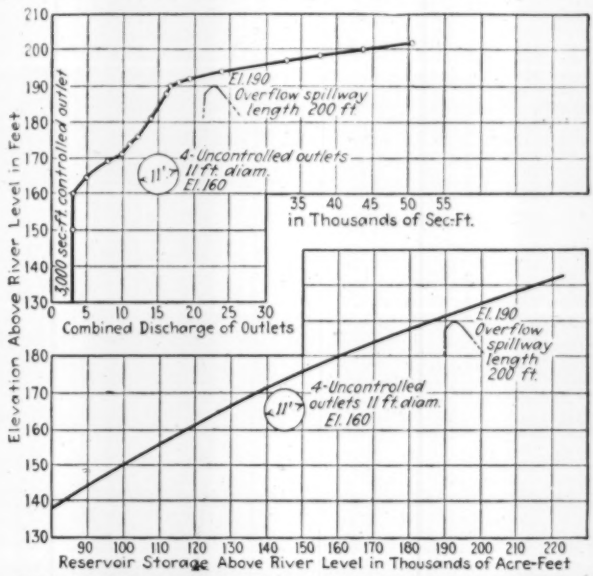
Graphics of Temporary Flood Storage

A Method of Calculation of the Balance Between Reservoir Inflow and Outflow During Floods, by Use of Mass Curves and a Polar Diagram of Outflow Rate

By Raymond A. Hill
Quinton, Code & Hill, Consulting Engineers, Los Angeles

IN THE design of dams for flood control, irrigation or power, the necessary capacity of spillways or other outlets is of prime importance, affecting both the safety and the cost of the structure. This capacity, as well as some other hydraulic features, is dependent on the amount of storage above normal full reservoir level or temporary flood storage. The latter must, therefore, be investigated. Obviously it acts to smooth out floods which enter the reservoir.

A number of methods have been developed for computing the effect of temporary flood storage on discharge. A step-by-step method depending on numerical computation entirely was used by the Miami Conservancy District: beginning with an assumed rate of outflow for a given time-interval, the storage was calculated by subtracting outflow from inflow, the water height was determined therefrom, and the assumed outflow rate was then checked from the spillway data, correcting by a second trial if necessary. In another method (*Engineering News*, March 21, 1912) a similar process is applied graphically to hydrographs of inflow and outflow, using the planimeter to measure water volume from the hydrographs. In a third method (*Engineering News*, June 13, 1912) the mass curve of inflow is used, and an outflow mass curve constructed in steps from trial calculations. These methods are typical of others. All are difficult of solution when the flow of the river varies rapidly and when there are several outlets at different levels.



FIGS. 1 AND 2—DISCHARGE AND CAPACITY CURVES
Above, outflow discharge curve. Below, Reservoir capacity curve.

In an effort to find a more direct and positive solution which could be used as easily with any combination of outlets and spillways as with a simple overflow spillway, the writer developed a graphical method about a year ago. It gives at least as accurate results as any other method and requires only a fraction of the labor.

In order to illustrate the flexibility of this method, a complicated flood-control problem is analyzed in detail. The reservoir is considered as being used for conservation up to 160 ft. above the river bed, so that normal full reservoir level is at this elevation. The inverts of four uncontrolled outlets, each 11 ft. in diameter, are set at this level, while at El. 190 there is an overflow weir 200 ft. in length. At all times of sufficient flow, 3,000 sec.-ft. are drawn out through outlets near the base of the dam. The flood considered in the calculations lasted only six days, and most of the runoff occurred in two days. The peak rate of inflow was 70,000 sec.-ft. and the total inflow about 180,000 acre-ft. The mass curve of inflow is shown in Fig. 4.

The first step in the calculations is to prepare a tabulation of basic data, such as that in the accompanying table, which gives the relation between the amount of water in storage, the elevation of the water surface, the discharge through each of the outlets and over the spillway weir, and the combined discharges. In some cases it is simpler to work directly from the reservoir capacity curve and from a curve of spillway and outlet discharges, such as those given in Figs. 1 and 2.

As stated before, any determination of the effect of temporary flood storage is based upon the existence of a definite relation between the stage of the reservoir and the rate of outflow from the reservoir. The key to this method of solution is a polar diagram which expresses this relation and which gives directly the slope of the mass curve of outflow corresponding to the stage of the reservoir. On a mass diagram the slope of the curve is the measure of the rate of flow, consequently the polar diagram must be constructed to the same time and quantity scales used in plotting the mass curve of the flood.

The polar diagram developed from the data given in the table is shown in Fig. 3. It is constructed as follows: From the table or from the curves in Figs. 1 and 2 it

RELATION BETWEEN STORAGE AND OUTFLOW

Reservoir Storage in Acre-Ft.	Elevation of Water Surface	Rate of Outflow in Sec.-Ft. Controlled	Uncontrolled	Overflow	Combined
100,000	150.0	3,000	0	0	3,000
117,000	160.0	3,000	0	0	3,000
120,000	161.5	3,000	550	0	3,550
122,000	162.7	3,000	900	0	3,900
126,000	164.7	3,000	1,900	0	4,900
130,000	166.7	3,000	3,000	0	6,000
134,000	168.6	3,000	4,300	0	7,300
135,000	169.3	3,000	5,000	0	8,000
138,000	170.5	3,000	6,200	0	9,200
139,000	171.0	3,000	6,800	0	9,800
145,000	173.7	3,000	7,800	0	10,800
150,000	175.8	3,000	9,000	0	12,000
155,000	177.9	3,000	9,800	0	12,800
162,500	181.0	3,000	10,900	0	13,900
165,000	182.0	3,000	11,200	0	14,200
175,000	186.0	3,000	12,450	0	15,450
180,000	187.9	3,000	12,950	0	15,950
186,000	190.0	3,000	13,600	0	16,600
188,000	190.8	3,000	13,700	500	17,200
190,000	191.5	3,000	13,950	1,350	18,300
191,500	192.0	3,000	14,100	2,200	19,300
195,000	193.3	3,000	14,400	4,600	22,000
197,000	194.0	3,000	14,600	6,200	23,800
200,000	195.0	3,000	14,900	8,600	26,500
205,000	196.8	3,000	15,300	13,700	32,000
206,000	197.0	3,000	15,400	14,600	33,000
210,000	198.4	3,000	15,700	18,900	37,600
215,000	200.0	3,000	16,100	24,500	43,600
220,000	201.7	3,000	16,500	31,000	50,500

will be seen that when the reservoir contains 200,000 acre-ft. the water surface is at El. 195.0 and the total outflow is 26,500 sec.-ft. From any convenient pole on the vertical axis a ray is drawn, whose slope above the horizontal is the same as that representing a rate of flow of 26,500 sec.-ft. on the mass diagram. Time is laid off on the horizontal axis and total reservoir storage on the vertical axis. A horizontal line is then drawn through the point on the vertical scale representing 200 000 acre-ft., intersecting the polar ray at L_1. Similarly, when the total amount in storage is 165,000 acre-ft., the discharge is 14,200 sec.-ft., represented by a ray of proper slope, which reaches the 165,000 acre-ft. ordinate at L_2. By the same process for a discharge of 6,000 sec.-ft., corresponding to 130,000 acre-ft. in storage, point L_3 is determined. Sufficient points are thus found to draw the polar curve shown. If a ray be now drawn from

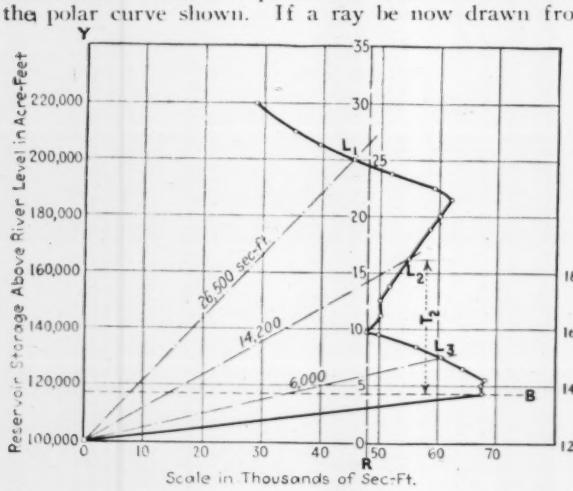


FIG. 3—POLAR RELATION BETWEEN STORAGE AND OUTFLOW

the pole to any point on this curve, its slope will represent the rate of outflow at the time the reservoir storage has the value shown by the vertical height of the chosen point.

For ease in determining the rate of flow represented by any polar ray, a vertical R-scale may be drawn, as shown. When second-feet and acre-feet are used, it is advisable to place this scale at an even number of days along the time scale from the pole.

This diagram may now be used in the determination of the outflow from the reservoir for any flood. The analysis of the particular flood considered is as follows:

The reservoir was at El. 160, the bottom of the uncontrolled outlets, at the beginning of the flood, and the water was held at this level by release through the controlled outlets until the rate of flow exceeded their capacity of 3,000 sec.-ft. Thus the initial storage was 117,000 acre-ft. On the polar diagram the horizontal line B is drawn through the point representing 117,000 acre-ft. on the storage scale. This line is henceforth used as a base line. If it be assumed for purpose of illustration that 67,000 acre-ft. had flowed into the reservoir during the flood and only 19,000 acre-ft. had flowed out, 48,000 acre-ft. would be temporarily stored in the reservoir above El. 160. In Fig. 4 the mass curve of inflow at that time would be above the mass curve of outflow by this amount of 48,000, or T_2. In order to determine the rate of outflow at the same instant, this distance T_2 is laid off vertically from the base line on the polar diagram, Fig. 3, to a point L_2 on the polar curve, and the ray drawn through L_2 is then parallel to the mass curve of outflow at that time and represents a rate of discharge of 14,200 sec.-ft. This is the rate corresponding to 165,000 acre-ft. of total storage, or to 48,000 acre-ft. of storage above the bottom of the uncontrolled outlets. This general method of determining the rate of outflow from the amount of storage is used in all further determinations.

On account of the slight difference between the rate of outflow and the rate of inflow during the first part of the flood, it will be assumed in the following description that the results at the end of the first seventeen hours have already been determined—namely, that the total inflow was 8,200 acre-ft. and that the total outflow was 3,000 acre-ft. (points N_1 and M_1 in the mass curves, Fig. 4). The level of the water in the reservoir has risen, therefore, and 5,200 acre-ft. of temporary flood storage has been used. At the end of seventeen hours the rate of outflow must be that corresponding to 122,200 acre-ft. of

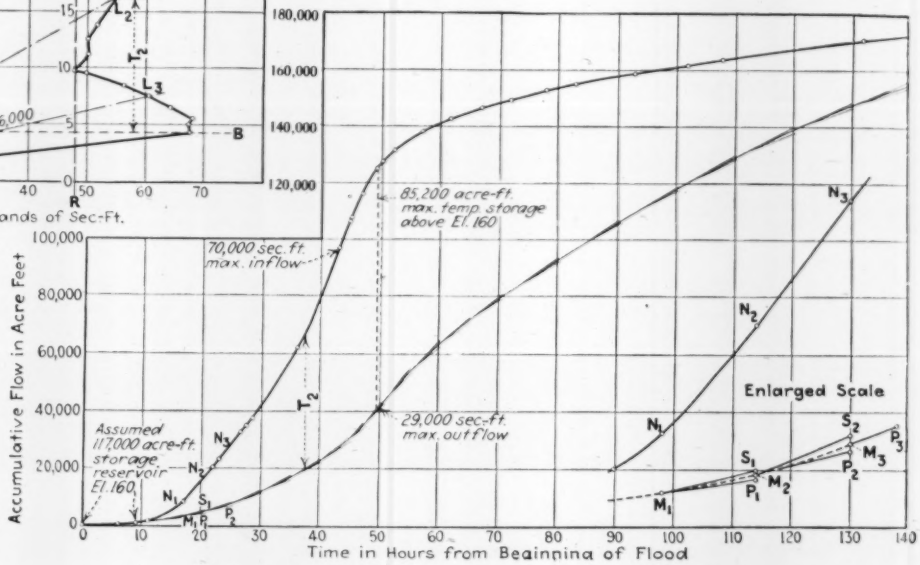


FIG. 4—MASS CURVES OF INFLOW AND OUTFLOW

total storage, and the equivalent ray on the polar diagram must be parallel to the tangent to the mass curve of outflow at that point, since the slope of the tangent to the mass curve at any point represents the rate of flow. The distance M_1N_1 between the mass curves of inflow and outflow, Fig. 4, is then laid off upward from line B in the polar diagram, and a ray is drawn through the point found on the polar curve; this ray gives the rate of outflow at that instant. The co-ordinates of the polar diagram and the mass diagram being parallel (usually plotted on the same sheet of co-ordinate paper), a line is drawn forward from M_1, the end of the mass curve of outflow, parallel to the ray just determined. This line is tangent to the mass curve at that point, and is extended a convenient distance M_1P_1 in Fig. 4, in this case to 21 hours.

If this rate of discharge had prevailed over the four-hour interval, the amount of temporary storage used would be that corresponding to the distance P_1N_2. With this amount in storage, however, the rate of outflow would be that represented by a ray with a greater slope due to the increase in the amount of water temporarily stored. The line M_1S_1 is then drawn parallel to this second ray. The true point M_2 on the mass curve of outflow therefore lies between the points P_1 and S_1. The distance M_2N_2 is then equivalent to the amount in storage above the level of the inverts, and the corresponding ray in Fig. 3 must be parallel to M_2P_2, the tangent to the mass curve of outflow at M_2.

In actual use the process is extremely simple, due to the fact that the two elementary rays diverge so slightly in a short distance that a point approximately midway between them can be selected without appreciable error. In Fig. 4 are shown all the rays required for an accurate solution of the problem considered.

Through the points M_1, M_2, M_3 . . . is drawn a smooth curve tangent to the rays M_1P_1, M_2P_2, M_3P_3 . . . which is the true mass curve of outflow from the reservoir. The rate of flow at any instant may be determined directly from the slope of the mass curve, and the amount of water in storage above normal full reservoir level at any time may be measured by the vertical distance between the mass curves of inflow and outflow.

In most cases a simple overflow weir is all that must be considered. Knowing the reservoir capacity at different levels, the corresponding polar diagram can be drawn in less than half an hour. The accumulative inflow can be computed and plotted as a mass curve in a few minutes, especially as discharge records are usually given as daily averages. After the polar diagram and the mass diagram have been prepared, the final solution can be carried through in an exceedingly short time. Actually, it has been found that one man can make in a day from five to ten complete solutions involving several combinations of outlets and several sizes of floods, and that errors are uncommon due to visual proof of each step in solution.

Fire-Hydrant Inspection by the Knoxville Water-Works

Annual Survey, With Repairs Where Needed, Made by Three-Man Crew With Tools and Truck Before Coldest Weather

By Frederick W. Albert
Engineer in Charge of Water Department, Knoxville, Tenn.

Based on a Statement sent to the technical press by Colonel Albert for the benefit of the water-works fraternity. The 1927-28 survey occupied 91 working days and cost $1,350, or an average of $14.83 per day, or 72.27c. per hydrant, including repairs. Inspection, oil, operation and adjustment cost 52c. per hydrant, exclusive of repairs. —Editor.

WITHIN the last four years the Water Department of Knoxville, Tenn., has conducted four fire-hydrant surveys. The time set is always in the autumn, in the hope and expectation that the survey will be completed before the advent of freezing weather, which usually hits Knoxville about Jan. 1. The first survey showed approximately 35 per cent of the hydrants in need of repairs. Many of these were two-nozzle hydrants, which were immediately replaced, so that now every hydrant has a standard steamer connection, in addition to two 2½-in. hose outlets.

The latest survey extended from Oct. 17, 1927, to Jan. 27, 1928 [prolonged by cold weather, mentioned later—Editor]. Of 1,868 fire hydrants inspected, 1,628, or 87.2 per cent, were found in first-class condition; 71, or 3.8 per cent, needed minor repairs (such as oil-hole screws, cap chains or chain hooks), whereas 148, or 7.9 per cent, needed major repairs (caps, valves, valve seats, upright valves and valve stems). A total of 21, or 1.1 per cent, needed replacement, since they were of an old make, for which repair parts could not be obtained.

Our procedure in a fire-hydrant survey is as follows: The inspector, a machinist and a laborer, with a truck, tools and materials, usually work from the outskirts to the center of the distribution system. Upon reaching a fire hydrant the inspector first takes his readings of the static and residual pressure. During these operations he learns whether or not the caps and other parts of the hydrant are holding, or whether they need new gaskets or other repairs. With the water shut off, the steamer cap is removed, and the inspector watches the draining of the fire hydrant. If the hydrant fails to drain, it is either marked for a "dig-out" or else the inspector and his men excavate around the hydrant, see that the drain holes are unstopped and place a new rock nest.

The stuff nut is then removed and the packing loosened and oiled, so as to provide a satisfactory cutoff of water leaking up around the stem. If, after shutting down the fire hydrant, the crew detects a constant flow, it is apparent that the valve is not properly shutting off. The hydrant is then opened and shut several times in the hope that any obstruction under the seat will be washed out. If this fails to stop the leak, the fire hydrant is put out of service by closing down the valve on the branch and the interior of the hydrant is then taken out. Valves, seat rings, stems and other parts are replaced as needed.

The cost of our 1927-28 fire-hydrant survey averaged 52c. per hydrant inspected, oiled, operated and adjusted. The cost of making minor and major repairs was $1.733 per hydrant repaired. The total cost of the survey (91 days of actual work) was $1,350. The average number of fire hydrants inspected and repaired per day was twenty, but two crews were used part of the time. The crews left the plant at 7:30, had half an hour for lunch and returned at 4:30.

The 1927-28 survey was interrupted by extreme cold weather in the early part of January, when the temperature dropped to —2 deg. F., the lowest since 1917, and a total of 103 hydrants were frozen. The department immediately put sixteen small crews of two or three men each on the work of inspecting and thawing out. The inspector took off the steamer cap and dropped a sounding lead into the barrel. Knowing the depth of the valve below the steamer outlet, the inspector could easily determine by the length of cord dropped whether the fire-hydrant barrel was free or frozen.

Locations of frozen hydrants were telephoned in to the central office, whereupon crews were sent out with thawing machines, or with kindling and oil. As a result the fire department, with more fire calls than usual, had very little difficulty in handling fires.

Concrete Paving-Base Control by Core Tests

Systematic Core Boring and Core Testing Stabilize Strength and Slab Depth in Akron Pavements

By E. A. Kemmler
Highway Engineer, Akron, Ohio

IN 1923 a core-drilling machine was purchased and put into use on paving work in Akron, Ohio. The slump permitted that year was 4 to 5 in., and the methods of testing the cores at the laboratory were crude. Thereafter, from year to year, the slump was reduced to 2 in. in 1927. Inspectors gradually became interested in the results of their supervision, and laboratory methods were perfected, so that by 1927 a system of control was in satisfactory working order. This progress is shown in

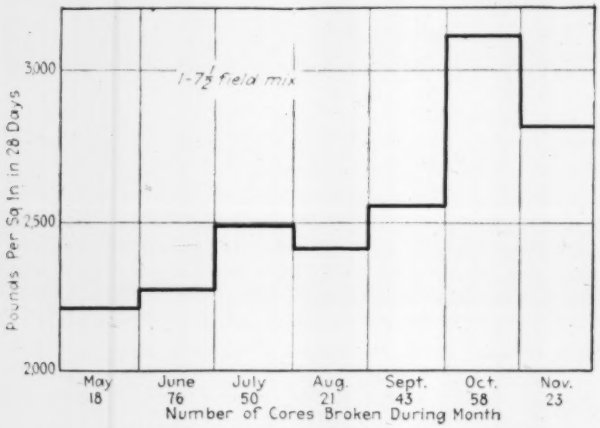


FIG. 1—MONTHLY VARIATION OF CORE STRENGTHS IN 1927

Fig. 3. During the five years of operation of the core drill there has been scarcely any trouble from depth deficiency; the tendency has been to excess. The 1927 record is as follows:

Required Thickness, In.	No. of Cores	Average Thickness, In.
6	226	6.27
7	199	7.53
8	23	8.24

The average crushing strength of the concrete (nominal 1 : 2½ : 5) in the 1927 cores was 2,542 lb. in 28 days, which is 742 lb. above the specification requirements. Cores were extracted at the rate of about one for each 150 ft. or 460 sq.yd. of base. The average cost per core, including laboratory tests, was $5.50. Although the peak strength obtained up to this time is not particularly high, the gain made from year to year is impressive.

That the increasing demands upon contractors for better proportioning and more careful selection and handling of materials has not caused any increase in contract prices is evidenced by the following average prices bid during the five years in which the system has been in operation:

Year	4-In. Brick With 6-In. Base	Asphalt With 7-In. Base
1923	$4.12	$3.65
1924	3.98	3.49
1925	3.97	3.14
1926	3.67	3.08
1927	3.57	3.05

With the improvement in depth and strength control, the next line of endeavor will be in the field of concrete base curing. The diagrams Figs. 1 and 2 show a distinct relationship between strength of concrete and seasonal atmospheric conditions. The method of curing the concrete heretofore practiced has been sprinkling with hose. The seasonal curve demonstrates that it has not been successful. The peak of strength occurred during October, when weather conditions were most favorable for natural curing. November was likewise a satisfactory month, but the low average temperature undoubtedly

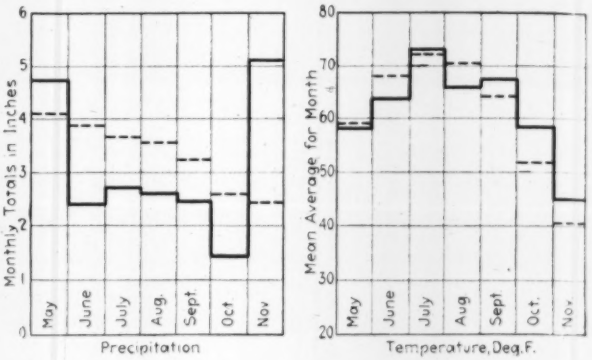


FIG. 2—TEMPERATURE AND PRECIPITATION VARIATION BY MONTHS IN 1927

Full lines indicate precipitation and temperatures during 1927. Dotted lines indicate averages of 40 years.

retarded the early gain in strength. The six months' test will probably tell a different story. The 1928 specifications provide for curing the concrete by ponding or covering with wet straw for seven to ten days. If our interpretation of the 1927 figures is correct, it is evident that we should have a more nearly horizontal seasonal curve in 1928.

The 4-in. cores have not been entirely satisfactory; the laboratory has now acquired a 200,000-lb. testing machine, and we are changing the drill to extract 6-in. cores. This should result in better specimens for the laboratory tests. The assistant engineers who are responsible for whatever progress has been made are F. W. Robertson, D. G. Cackler and M. D. Wilson.

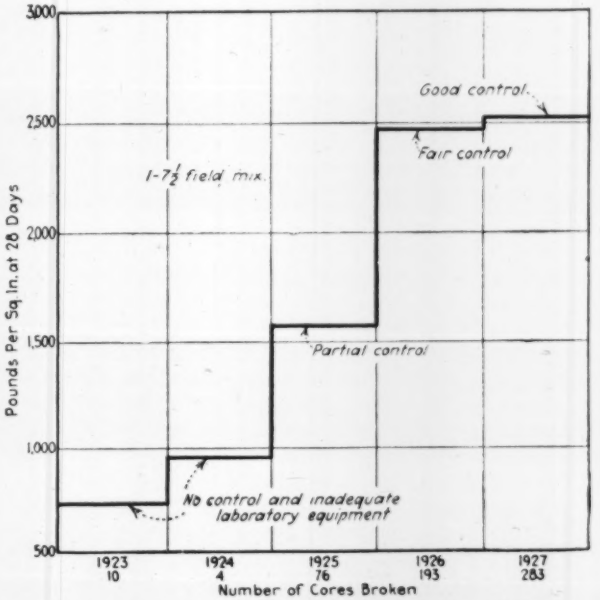


FIG. 3—INCREASE IN STRENGTH OF CONCRETE BASE IN FIVE YEARS

Locating Railway From Mexico City to Pacific Coast

Alternative Direct Lines Have Several Summits—Long Indirect Line Is Mainly Down Grade

By G. H. T. Shaw
Consulting Engineer, Chicago

FOR many years the government of Mexico has desired to effect rail communication between Mexico City and the port of Acapulco, on the Pacific Coast. Some of the reasons for considering this railway project as of importance to the nation are: (1) The bay of Acapulco is the best harbor on either of the Mexican coasts and one of the best in the world; (2) the railway is desirable from a military standpoint, both in regard to possible foreign invasion and for the quelling of local uprisings; (3) it would reduce the distance and expense of transportation between the rich central plateau and a Pacific port; (4) it would stimulate industrial development in one of the richest sections of the country; (5) a route through the Balsas Valley, now entirely without rail service, would develop vast natural resources in agriculture, mining and timber.

Present rail communication between Mexico City and the Pacific Coast is furnished by two lines: (1) The National Railways of Mexico, through Guadalajara to the port of Manzanillo, 968 km. (601 miles); (2) the Mexican Railway to Cordova, 318 km. (198 miles) and then the National Lines to Salina Cruz, 509 km. (316 miles), a total of 827 km. (514 miles). Both have standard-gage roads with ruling grades of 3 and 4 per cent.

As Mexico City (El. 2,239 m., or 7,346 ft.) is on the Gulf slope, a railroad to the Pacific Coast must surmount the Continental Divide. A straight line to Acapulco (approximately 300 km., or 186 miles) would cross the Continental Divide at about 50 km. (31 miles) southwest of the city and then the high Sierra Madre del Sur range paralleling the coast line. The lowest pass for an exit from the valley of Mexico for a railroad to Acapulco is near the village of Ozumba (El. 2,400 m., or 7,874 ft.), and is utilized by the narrow-gage line of the National Railways toward Cuautla. Another pass, La Cima (El. 3,016 m., or 9,894 ft.) is occupied by the standard-gage line of the National Railways to Cuernavaca and the Balsas River. The same company's narrow-gage road to Toluca and Acambaro crosses the divide at another pass, also called La Cima (El. 3,060 m., or 10,039 ft.) near Salazar. Other passes present no decided advantages. The first two passes named lead to the drainage area of the Balsas River, the third leads to that of the Lerma River, both discharging into the Pacific Ocean.

In general, three major routes are indicated for rail connection between Mexico City and Acapulco. These three locations are shown on the accompanying map and profiles.

1. Mexico City to the pass near Ozumba, thence by the valleys of the Cuautla, Amacusac and Balsas rivers and up the Atliaca River to cross the coast range at the pass of Cumbre de Atliaca; thence to Chilpancingo, through the pass called Cima de Valades and on to Acapulco 518 km. (322 miles). This route has been surveyed for a maximum compensated grade of 2 per cent, with 10 deg. curves. Total ascent, 1,073 m. (3,520 ft.); total descent, 3,312 m. (10,866 ft.).

2. From Mexico City by the existing line through the pass of La Cima to Iguala. Then by a new line to the Balsas River at Mexcala, crossing the Sierra Madre del Sur range through the pass of Tierras Prietas and joining the first location at Chilpancingo. For this line the total distance is 493 km. (306 miles), with 3 per cent compensated grades and 10 deg. curves; total ascent, 2,333 m. (7,654 ft.); total descent, 4,572 m. (15,000 ft.).

3. From Mexico City to the La Cima pass, near Salazar. The elevation of this pass on the National Railways is 3,060 m. (10,039 ft.), but the writer's survey, using a deep cut, crosses at El. 3,018 m. (9,901 ft.). Thence to Toluca and Tulaltongo, through the La Estancia pass and with practically no adverse grades to Pungarabato. It follows the left bank of the Balsas River to Zacatula and then parallels the coast, with but very little adverse gradient, to Acapulco, a total distance of 1,045 km. (649 miles). This route has been surveyed for a standard-gage road with a 2 per cent compensated maximum gradi-

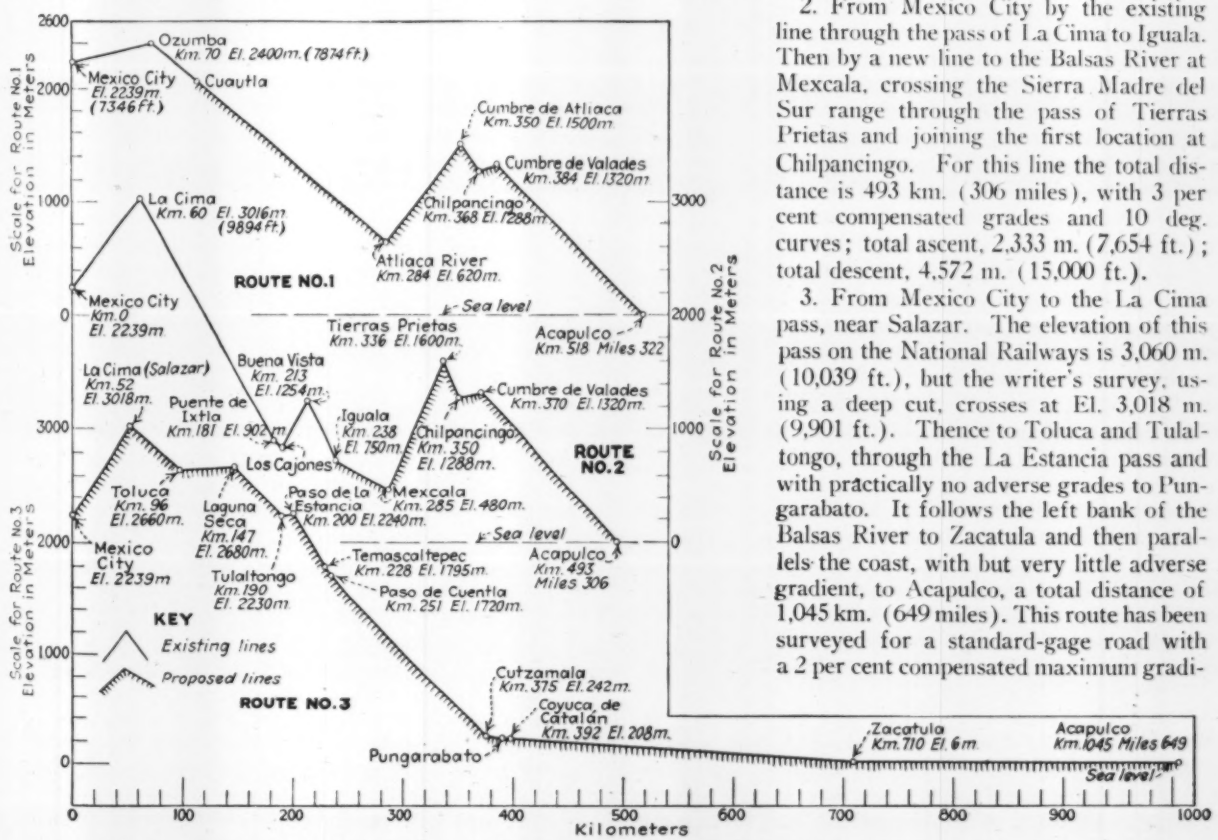

Scale for Route No.1 Elevation in Meters
Scale for Route No.2 Elevation in Meters
Scale for Route No.3 Elevation in Meters
Ozumba Km.70 El. 2400m. (7874 ft.)
Mexico City El. 2239m. (7346 ft.)
Cuautla
Cumbre de Atliaca Km.350 El. 1500m.
Cumbre de Valades Km.384 El. 1320m.
Chilpancingo Km.368 El. 1288m.
La Cima Km.60 El. 3016m (9894 ft.)
Atliaca River Km.284 El. 620m.
ROUTE NO.1
Mexico City Km.0 El. 2239m.
Sea level
Tierras Prietas Km.336 El. 1600m.
Acapulco Km.518 Miles 322
La Cima (Salazar) Km.52 El. 3018m.
Buena Vista Km. 213 El. 1254m.
Puente de Ixtla Km.181 El. 902m.
Iguala Km.238 El. 750m.
Chilpancingo Km. 350 El. 1288m.
Cumbre de Valades Km.370 El. 1320m.
ROUTE NO.2
Los Cajones
Toluca Km. 96 El. 2660m.
Laguna Seca Km.147 El. 2680m.
Paso de la Estancia Km. 200 El. 2240m.
Mexcala Km. 285 El. 480m.
Sea level
Mexico City El. 2239m
Tulaltongo Km.190 El. 2230m.
Temascaltepec Km.228 El. 1795m.
Acapulco Km. 493 Miles 306
Paso de Cuentla Km.251 El. 1720m.
KEY
Existing lines
Proposed lines
ROUTE NO.3
Cutzamala Km.375 El. 242m.
Coyuca de Catalan Km.392 El. 208m.
Pungarabato
Zacatula Km.710 El. 6m.
Acapulco Km.1045 Miles 649
Sea level
Kilometers

FIG. 1—PROFILES OF ALTERNATIVE ROUTES

ent to the Balsas River and 1 per cent from there to Acapulco. Total ascent, 809 m. (2,654 ft.); total descent, 3,048 m. (9,999 ft.).

The first location has the advantages of the lowest summit, low maximum grade, a total length of only 518 km. (322 miles) and a large proportion of water-course location with consequently few adverse grades. The second location has these advantages: 238 km. (148 miles) of existing railroad, leaving only 255 km. (158 miles) for construction; it is the shortest line and practically parallels the old pack trail, along which are centers of population. Although the third location has a much greater length than the others, it has advantages in low grades and little adverse grade in its descent to the coast, while it would open up a territory that has great timber, mining and agricultural resources, but is badly in need of railway facilities. In the writer's opinion, from many years of experience in studying railroad projects in Mexico, this last location seems the most advantageous for national betterment.

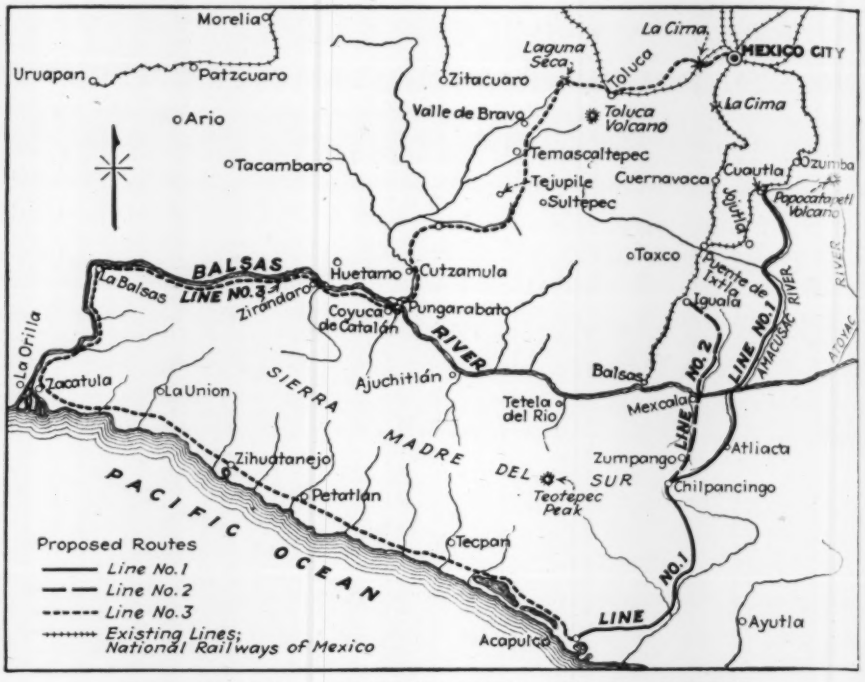


FIG. 2—PROPOSED LOCATIONS; MEXICO CITY TO ACAPULCO

In cost per mile the second route would be the most expensive and the third would be the cheapest. For steam operation the first is preferable, but on all the routes there is water power that might be developed for electrical operation on the heavier grades. During 1927 an automobile highway was completed, except for major bridges, between Mexico City and Acapulco, but this does not relieve the necessity for rail connection.

In the preparation of this article the writer not only has used his own personal notes and information furnished by the Department of Communications of the Mexican government but is also indebted to a compilation of data regarding these routes, written by Leon Salinas, engineer, and published in the *Revista Mexicana de Ingenieria y Arquitectura* of August, 1924.

Passaic Sewerage District Act Simplified

The act creating the Passaic Valley Sewerage District (New Jersey) has been modified so that any municipality may lease a part of its contract capacity in the sewer and disposal works to other municipalities or to industrial concerns either within or outside the district, instead of having to get the approval of all the other cities and towns in the district. The district comprises fifteen municipalities, with Paterson at its upper and Newark at its lower end. It is expected that West Paterson and Haledon will enter through Paterson and a part of Saddle River Township through Garfield; also that several industrial plants will be connected with the trunk sewer, pending the construction of treatment works. All leases must be approved by the commission and may be terminated by either municipal signer on one year's notice.

Long Highway Tunnel in Zion National Park

Construction is now well under way on a national park highway project in Zion National Park, Utah, which involves road construction of a spectacular and unusual nature. The project as a whole involves 8.75 miles of new road, which will be part of the highway connecting Zion Park with the north rim of the Grand Canyon of the Colorado. Of this total mileage, contract for 5.02 miles was awarded to the Nevada Contracting Company last September for $607,000 and the work is scheduled for completion in August, 1928. This portion includes a 5,582-ft. tunnel with six galleries or openings in the face of the adjacent cliff, commanding a view across the valley 1,000 to 1,200 ft. below.

The face of the cliff behind which this tunnel is being built is a rock wall rising vertically several hundred feet from talus slope, which it actually overhangs in some places. The tunnel location was influenced by the need for easy curves, reasonably short distances between openings and the desirability of bringing the road to the cliff face where openings were made. These requirements were complicated by the extreme irregularity of the cliff face and the fact that in making surveys it was impossible to occupy any of the points along the face of the cliff. Location of the galleries and other survey work had to be done wholly by triangulation from lower levels on the opposite side of the canyon, without even getting rodmen to the points located on the cliff.

The tunnel roadway is being made 22 ft. wide on a grade of 5 per cent, and the greatest length between galleries will be 1,100 ft. It has not yet been decided whether or not electric lighting will be necessary. The remaining 3.73 miles of road on the project, which will probably be let this summer, includes several tunnels that will total 1,626 ft. in length.

The work is being done under the general supervision of the U. S. Bureau of Public Roads.

Design of the Owyhee Irrigation Dam

Strikingly New Features in 405-Ft. Arch-and-Gravity Dam to Be Built in Oregon—Deep Fracture Fault in Floor Rock May Require Cutoff 100 Ft. Below Base—Floating Ring-Shaped Spillway Gate—Elevator in Dam

By J. L. Savage
Chief Designing Engineer, U. S. Bureau of Reclamation, Denver

AT THE "Hole in the Ground" reservoir site on the Owyhee River in Oregon, once the rendezvous of cattle rustlers, the Bureau of Reclamation plans to build the principal storage reservoir for the irrigation of the Owyhee reclamation project, embracing 120,000 acres in eastern Oregon and western Idaho. The reservoir, formed by a dam 21 miles southwest of Nyssa (the nearest point on the main line of the Union Pacific Railroad), will have a total capacity of 1,120,000 acre-ft. and an available capacity of 715,000 acre-ft. To create this large storage a dam of exceptionally great height was required. The design of this structure not only involved most elaborate preliminary studies and thorough analyses but called for the development of a number of features new in dam construction.

The dam is to be built of concrete and will be of combined arch-and-gravity type. It will be 390 ft. high above the general elevation of the foundation, 405 ft. high at the maximum section and about 520 ft. high above the lowest concrete in the foundation cutoff. In storing 715,000 acre-ft. of available water the dam will raise the water surface 234 ft. above river elevation for diversion into the main canal tunnel and will create an average head of 275 ft. for the incidental development of power from water which has to pass the dam for prior-right use.

Geological and Engineering Investigations

The recent unfortunate failure of the St. Francis dam in California justifies special mention of the extensive geological and engineering investigations that preceded the approval of the site and designs for the Owyhee dam. Nearly $100,000 has been expended during the past three years for these investigations, which included the drilling of 73 test holes at the site with nearly 1½ miles (7,800 lin.ft.) total depth. Five different geological examinations of the reservoir and dam site were made by Dr. F. L. Ransome, professor of economic geology at the California Institute of Technology, Dr. Warren D. Smith, professor of geology at the University of Oregon, and Kirk Bryan, of the U. S. Geological Survey. Three consulting engineers, A. J. Wiley, of Boise, D. C. Henny, of Portland, and Charles H. Paul, of Dayton, also passed on the site and reviewed the designs and estimates.

Geology of Reservoir Site—With water depths over the reservoir floor reaching nearly 350 ft. at the dam and with 270 ft. of dead storage in the reservoir at all times, the geology of the basin to be flooded was obviously important. The deep parts of the reservoir are in pre-basaltic tuff, satisfactory for water retention, while the upper part is in practically watertight formations consisting of conglomerate, sandstone, shale and tuff. Columbia River basalt in the reservoir floor near the mouth of Dry Creek is the one material presenting any real doubts. But while this rock may offer passage to water, the flow would have to travel many miles to an outlet, with various possibilities of stoppage, and the geological opinions indicate that no considerable leakage will occur at this point. The reservoir site has therefore been pronounced suitable for the retention of water.

Geology of Dam Site—The test borings show the river fill at the dam site to consist of sand, gravel, cobbles and boulders to a maximum depth of 60 ft. below normal river surface, overlying glassy rhyolite similar to that in the canyon walls. This rhyolite extends to a minimum depth of 170 ft. and a maximum depth of 215 ft. below river surface at the dam; it is bedded on 20 ft. of pitchstone agglomerate, and this in turn is bedded on tuff of unknown depth. Several modifications of the rhyolite occur: felsite, red felsite, pitchstone or obsidian, felsite breccia and pitchstone agglomerate. The rhyolite is believed to be an intrusion into tuff while still covered with water and before complete cementation of the tuff beds. It is a hard, durable rock, and while the "pitchstone agglomerate" variety is softer, it too was declared by Dr. Ransome to be suitable as an abutment rock.

Shattered Fault Zone—A shattered zone or fault line crosses the site at about the center of the canyon. First disclosed by the core drilling, this shattered zone was later exposed by an open test pit where it leaves the river canyon at a point about 1,450 ft. downstream from the dam. The shattered zone is bounded by two thin seams of clay gouge about 10 ft. apart. The thickness of the gouge is only a fraction of an inch, which, according to Dr. Ransome, indicates a probable movement of less than

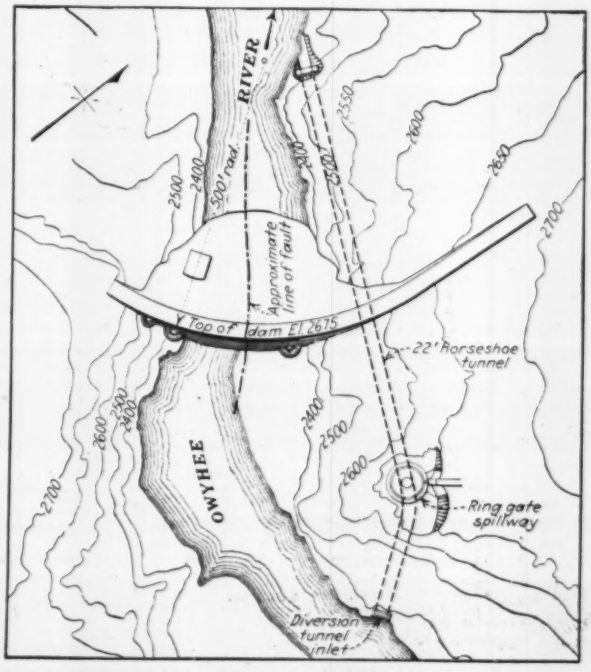


FIG. 1—SITE OF OWYHEE DAM

100 ft. along the fault. Between the gouge seams is shattered rhyolite, fragments generally under 6 in. in their longest dimension and occasionally reaching 14 in. The fragments, although minutely fractured, are not displaced or rotated from their original relative positions, and there are no large interstices between the fragments. However, the fragments are not cemented together, and the entire fault filling is easily excavated with pick and shovel. The material in the fault would allow water to pass through it, and must therefore be sealed by a cutoff. The geological forecast is that the fault probably extends through the full depth of the rhyolite, but not into the underlying agglomerate and tuff, which are less brittle and would not shatter from the fault movement as did the rhyolite. This opinion is borne out by the drill cores.

A cutoff is to be constructed through the full depth of the rhyolite but will probably terminate at either the agglomerate or the tuff. If the open excavation shows it to be necessary, this cutoff will be extended to the full width of the base of dam.

Study of Alternative Designs

Preliminary designs and estimates were prepared for five different alternative types: (1) a light arch section, (2) an intermediate arch section, (3) a heavy arch or arch-and-gravity section, (4) a straight gravity dam, and (5) a slightly curved gravity dam. The general dimensions and total excavation and concrete quantities for these alternative studies are summarized in the accompanying table.

The heavy arch type is in reality a section which if straight and not subject to uplift pressures would be as safe as a gravity dam; its pressure resultant would pass through the downstream third-point and the sliding factor would be 0.65 or less at all elevations. While this heavy arch dam is in no sense a gravity section, it has been termed an arch-gravity dam for the reason that the loads are carried partly by arch action and partly by gravity action. This designation is in a way misleading, as the two lighter arch sections were also designed with the loads carried partly by arch action and partly by gravity action. The straight gravity section and the slightly curved gravity section were designed strictly as gravity dams, all of the water load being carried by gravity action.

SUMMARY OF EXCAVATION AND CONCRETE QUANTITIES FOR FIVE ALTERNATIVE SECTIONS OF DAM
(Top Thickness 20 Ft. in All Designs.)

Dimensions and Quantities	Alternative Sections of Dam: Light Arch	Intermediate Arch	Heavy Arch	Straight Gravity	Curved Gravity
Upstream top radius...	350	375	500	.....	1,000
Max. base thickness, ft.	130	160	242	296	296
Excavation and concrete:					
Earth and loose rock, cu.yd.	107,000	123,000	151,000	159,000	161,000
Solid rock, cu.yd.	*85,000	*107,000	76,000	77,000	83,000
Concrete, cu.yd.	363,000	474,000	500,000	550,000	570,000

*The apparent inconsistency in the rock excavation quantities for the light and intermediate arch sections as compared with the heavy arch and gravity sections is due to the greater assumed excavation to provide radial abutments for a considerable portion of the abutment areas.

Data of Analysis—In all of these studies, including both the arch and gravity sections, uplift pressures have been assumed to act over the whole area of the base varying from full hydrostatic pressure at the upstream face to one-half hydrostatic pressure at the drainage wells and diminishing uniformly thence to zero or tailwater pressure at the downstream face. While it is realized that uplift pressures of full hydrostatic intensity cannot be effective over the whole area of the base at any point, the foregoing assumption has been arbitrarily made to insure an ample allowance for uplift pressures. The vertical components of water pressure on the upstream and downstream faces have likewise been included in the cantilever loads.

The effects of shear and moment deflections of the foundation and of convergence of the sides were included in the cantilever studies, while the arch studies took account of shear, rib shortening and abutment deflections due to thrust, moment and shear. In connection with the arch studies relative to abutment deflections, a number of 5x5x9-in. samples of the foundation rock (felsite) were tested, showing a modulus of elasticity of 8,555,000 lb./in.² (two specimens), Poisson's ratio 0.235 (two specimens) and crushing strength 9,040 lb./in.² (four specimens). The

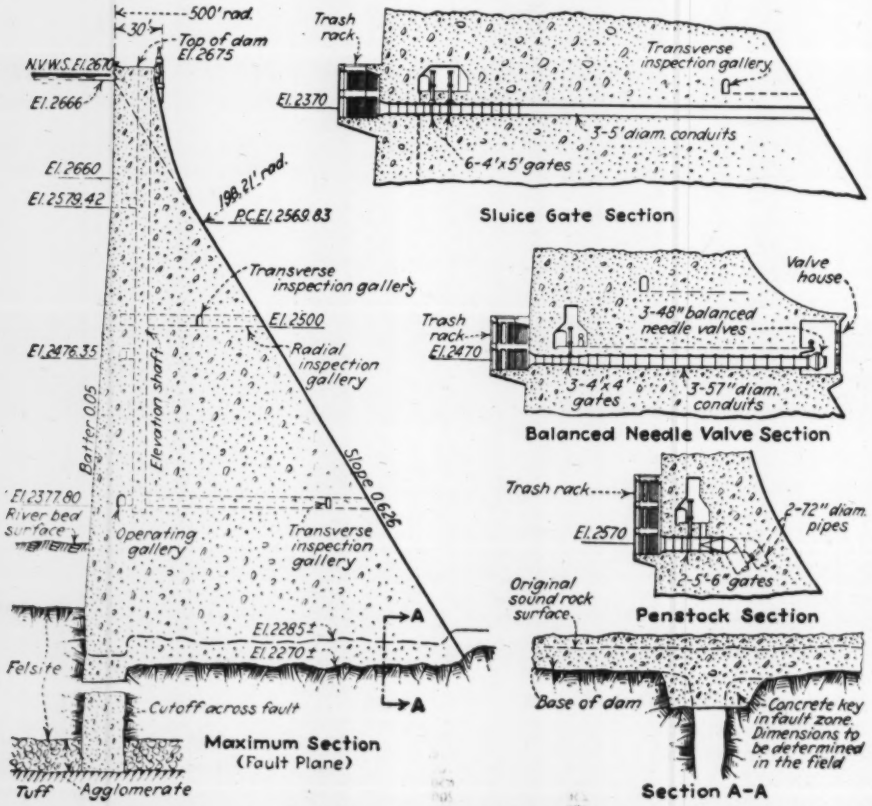


FIG. 2—MAXIMUM SECTIONS AND ARRANGEMENT OF CONDUITS

formulas for calculating the yielding of foundation and abutments are taken from the paper "Ueber die Berechnung der Fundament-deformationen," by Dr. F. Vogt. The preliminary computations include the effect of yearly temperature changes, while the final studies will include consideration of the effect on the stresses of the setting heat remaining in the concrete at the time of grouting the radial contraction joints. It is also intended to study in the final computations the effect on the stresses of saturation of concrete.

With no allowance for the effect of the heat of setting remaining in the arches at the time of grouting the radial contraction joints, the preliminary computations indicate that about three-quarters of the water load will be carried by arch action and one-quarter by gravity (cantilever) action. While the maximum stresses have not been finally determined, the preliminary computations indicate moderate stresses in all parts of the structure, with the arch stresses exceeding the cantilever stresses by a considerable amount due to the greater proportion of the load being carried by the arches.

It is planned to grout the vertical contraction joints in separate units while the arches are under some initial water load, in order to distribute the full water load between the arches and cantilevers and reduce the arch stress. Probably this improvement in the load distribution can be accomplished in the normal operation of filling the reservoir, but if necessary the filling of the reservoir may be begun before the dam is completed, and on the other hand the grouting may be deferred, in which case the closing plug of the diversion tunnel will not be placed until this work is ready to begin.

The load distribution between the arch and cantilever elements was calculated by the trial load method, which has been in use by the Bureau of Reclamation engineers for several years. The distribution between arches and cantilevers was carried to close agreement, using loads in increments of 100 lb. The maximum disagreement between arch and cantilever deflections is about 5 per cent, with the average over the whole structure less than 2 per cent.

Arch-Gravity Type Adopted — As the comparison showed the heavy arch type to be most efficient, it was adopted. However, the dam as finally designed differs somewhat from the preliminary design in dimensions. The top thickness will be 30 ft. and the bottom thickness at maximum section about 264.5 ft. The upstream face will have a batter of 0.05 : 1 and the downstream face will be generally on a slope of 0.626 : 1.

Special Features of the Structure

Outlets—Three 4x5-ft. sluice-gate outlets are provided at river level, each outlet being controlled by two 4x5-ft. cast-iron sluice gates, operated by oil pressure cylinders from a gallery in the dam. The outlet conduits will be lined with cast iron from a point 20 ft. upstream from the upstream gate to a point 30 ft. downstream from the downstream gate. The rectangular cross-section of outlet conduits changes to 60-in. circular cross-section within a 10 ft. length of transition section beginning 4 ft. downstream from the downstream gate. The usual air vent pipes are provided to prevent vacuum below the gates at partial openings. The protective trash-rack structure consists of semi-cylindrical structure 36 ft. in diameter and 24 ft. high constructed on the face of the dam. The rack bars will be 6x$\frac{3}{8}$-in. steel bars spaced 6 in. on centers.

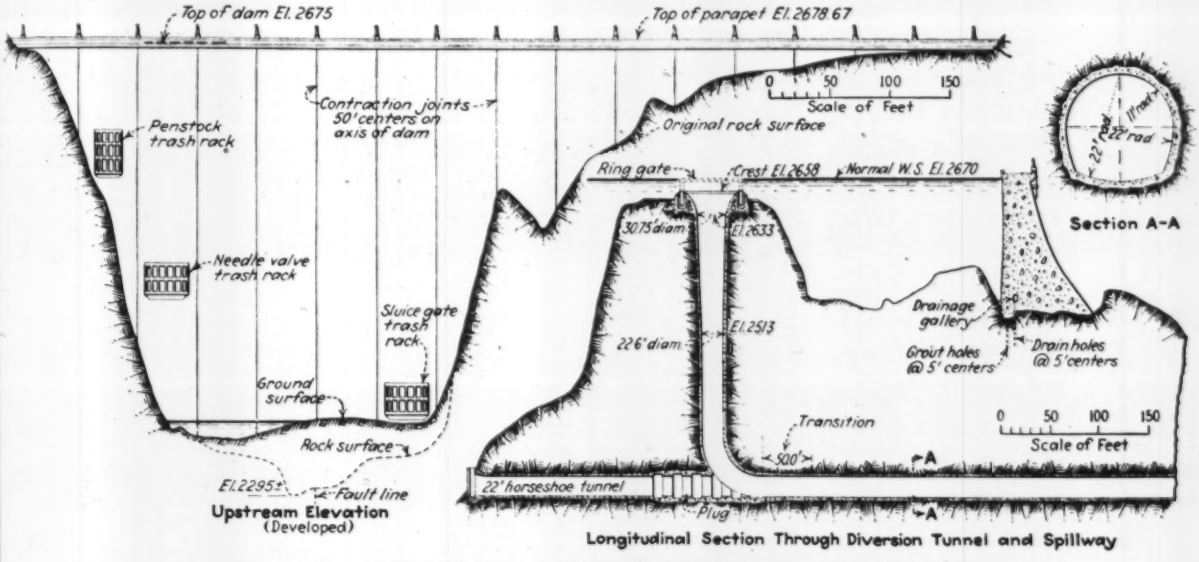


FIG. 3—ELEVATION OF DAM AND SECTION THROUGH TUNNEL SPILLWAY

Three cast-iron-lined conduits controlled by 48-in. balanced needle valves and a 4x4-ft. high-pressure emergency gate provide for passing prior-right water and controlling the reservoir. The emergency gates will be operated by oil. The needle valves, mounted on the downstream end of the outlet conduits, are protected by a concrete valve house constructed on the downstream face of dam. Trash racks of the same bar size and

SUMMARY OF TRASH-RACK VELOCITIES

Head on Center of Outlet in Feet	Sluice Gate Outlet — Outlet Capacity, Sec.-Ft.	Sluice Gate Outlet — Velocity Through Racks, Ft. Per Sec.	Needle Valve Outlet — Outlet Capacity, Sec.-Ft.	Needle Valve Outlet — Velocity Through Racks, Ft. Per Sec.
25	.....	...	1,000	0.9
50	2,300	2.1	1,400	1.3
75	.....	...	1,700	1.6
100	3,300	3.0	2,000	1.8
150	4,100	3.8	2,400	2.2
200	4,650	4.3	2,800	2.6
250	5,200	4.8	.....	...
300	5,700	5.3	.....	...

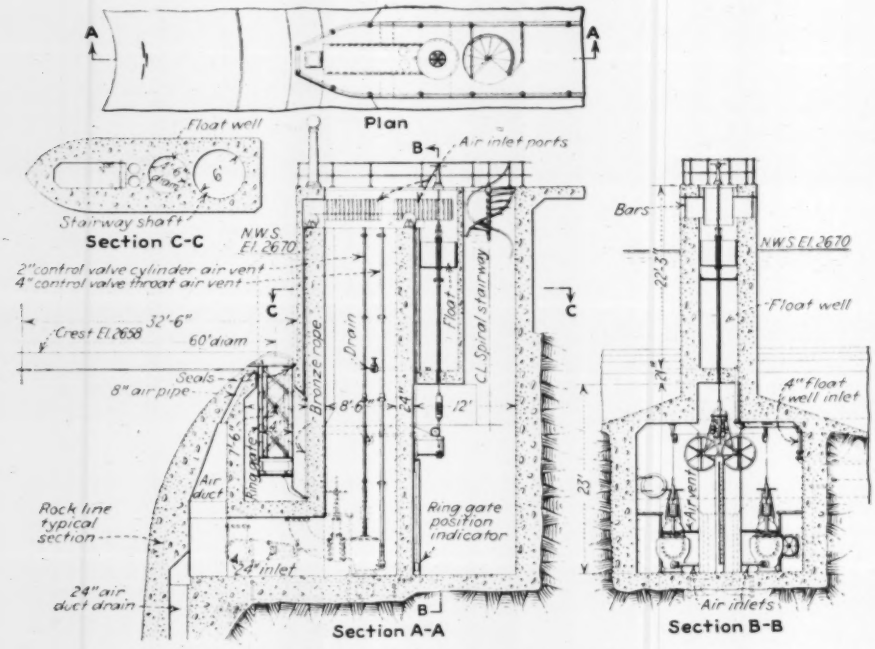


FIG. 4—DETAILS OF RING GATE OF SPILLWAY

spacing already noted are placed in a semi-cylindrical structure 36 ft. in diameter and 24 ft. high.

Provision is made in the design of the dam for the future installation of a power plant to utilize the flow of prior-right water. Two 6-ft. conduits are provided, each conduit being controlled by a 5x6-ft. cast-iron emergency gate operated by oil pressure. These outlets will be

COMPARISON OF DIVERSION TUNNEL CAPACITY WITH RIVER DISCHARGES

Height of Cofferdam, Feet	Tunnel Capacity, Sec.-Ft.	Tunnel Capacity Exceeded
43	12,000	23 years in 26
58	15,000	7 years in 26
75	18,000	3 years in 26

Note—Freeboard of 5 ft. allowed in cofferdam height. River-flow data obtained from 26-year record.

located 100 ft. below high water and 20 ft. below the bottom of irrigation storage. The trash rack at the penstock openings are set in 3¼-in. spacing.

Trash-Rack Structures—All trash-rack structures are designed for a head of 40 ft. with normal stresses in the reinforced concrete and yield-point stress in the bars; if the racks were blocked, the bars would therefore fail before the reinforced-concrete structure would be damaged. Submerged trash racks of this general type have been used for many years under similar conditions without plugging, and when designed with low rack velocities, are usually found free from trash when unwatered.

The various trash racks to be constructed for this dam have been designed for low velocities, as will be noted from the accompanying tabulation, in which values are based on 100 per cent gate openings.

Diversion Tunnel—A 22-ft. horseshoe tunnel 1,005 ft. long is to pass the river flow during construction and serve the permanent spillway subsequently. Headwall grooves are provided in the intake structure for making temporary closure of the tunnel, and keyways upstream from the spillway shaft will hold the concrete plug for final closure. The table shows discharge capacities at different heights of cofferdam.

Ring Gate Spillway—A radically new spillway was designed for the Owyhee dam. It is a circular spillway, controlled by a ring gate. Patent application has been made for the gate and some of the control arrangements. The spillway is the lip of a funnel formed at the head of a shaft discharging into the diversion tunnel 240 ft. below the tunnel intake. The funnel is surrounded by a structural steel ring gate 12 ft. high and 60 ft. in peripheral length, constituting a floating crest. It is planned to build the gate in segments joined in the field. For control of the gate a needle valve is used. This arrangement follows closely the recent drum gate installations of reclamation dams.

Minor Features—An elevator for supplies and repair parts of the gates is one of the innovations in dam practice required by this great structure. Since many of the valves are operated from interior galleries and require large and heavy parts, this auxiliary was essential. There are, of course, other means of access, as spiral stair shafts and galleries.

Grout holes 5 ft. apart along the upstream cutoff, which may have to be drilled to depths of 25 to 150 ft., are to be provided for grouting the rock. The fault zone area of the base will probably require unusually deep grouting. Drainage holes will be drilled 20 to 40 ft. deep, from the spiral stairway shafts and operating galleries. By doing this work after the dam is partly built the drainage system will not be affected by the grouting operations. Finally the vertical contracting joints are to be grouted, by way of embedded pipes and conduit fitting as in the Gibson dam, now under construction by the Bureau of Reclamation on the Sun River project in Montana.

Principal Construction Items

The construction of the Owyhee dam will involve the following principal items of work:

1.	Excavation: earth and loose rock, cu.yd.	135,000
2.	Excavation: solid rock, cu.yd.	70,000
3.	Excavation: all classes, spillway shaft, cu.yd.	15,500
4.	Excavation: all classes, diversion and spillway tunnel, cu.yd.	35,000
5.	Drilling grout holes, lin.ft.	11,000
6.	Drilling drainage holes, lin.ft.	7,000
7.	Concrete in dam, cu.yd.	490,000
8.	Concrete in fault zone, cu.yd.	17,600
9.	Concrete in spillway inlet and outlet, spillway shaft and tunnel lining, cu.yd.	12,000
10.	Concrete in needle valve house, trash-rack structures, etc., cu.yd.	825
11.	Placing reinforcing steel, lb.	675,000
12.	Installing pipe and fittings for grouting radial contraction joints, lb.	115,000
13.	Installing and painting gates and valves with appurtenant metal work, lb.	1,760,000
14.	Installing and painting structural steel ring gate with appurtenant metal work, lb.	440,000
15.	Installing and painting trash-rack steel, lb.	183,000

Construction Railroad—At present the General Construction Company, of Seattle, Wash., is building a con-

struction railroad 24 miles long to the site from Dunaway Siding on the Homedale branch of the Oregon Short Line Railroad. It is to be operated by the contractor for the dam. Electric power at 2,300 volts will be transmitted by the Idaho Power Company from Black Canyon dam over the company's lines to the Ontario-Nyssa substation and thence over a government 66,000-volt single-circuit wood-pole transmission line now being built. Excellent sand, gravel and cobbles are found in the vicinity of Dunaway Siding. The material is to be washed and separated into sand, three sizes of gravel and cobbles. Cobbles up to 8 in. in diameter will be permitted in the massive concrete in the dam.

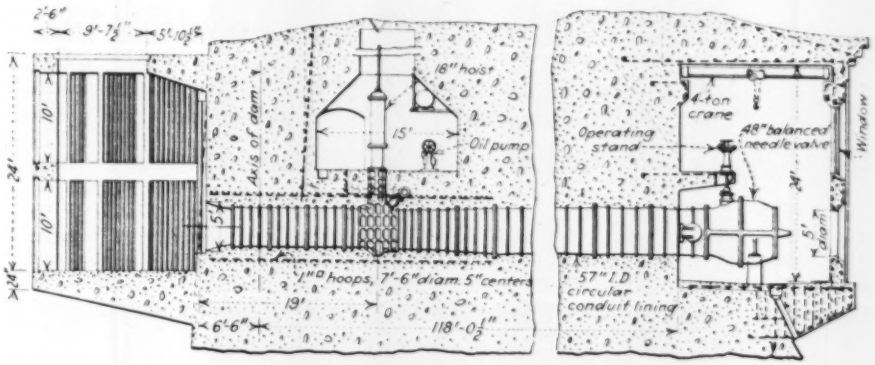


FIG. 5—NEEDLE-VALVE OUTLETS

Bids on the construction of the Owyhee dam will be opened early in June and the contract will be let before the end of the current fiscal year. The field engineering and construction work will be under the general supervision of F. A. Banks, construction engineer, of Nyssa, Ore. The designs were prepared under the immediate direction of B. W. Steele, engineer on dams, of the Denver office. The principal designing work for the Bureau of Reclamation is done in the Denver office under the general direction of the writer. All engineering and construction work is under the general supervision of R. F. Walter, chief engineer, with headquarters at Denver, and all activities of the bureau are under the general charge of Dr. Elwood Mead, commissioner, with headquarters at Washington.

First-Order Leveling in New England During 1927

WITH the completion of 841 miles of first-order leveling in Maine, Massachusetts, New Hampshire and Vermont during the past year the U. S. Coast and Geodetic Survey has extended the first-order level net in the New England states until it now totals 2,406 miles and fixes the elevations of 1,701 points. Of the total leveling, 2,163 miles were run by the U. S. Coast and Geodetic Survey, 200 miles by the Massachusetts Topographic Commission and 43 miles by the engineering department of Columbia University. When considered with the leveling of the Geodetic Survey of Canada, practically no place in this area is more than 50 miles from a permanent bench mark.

Two adjustments of this leveling have been made, one to test its accuracy and the other to provide elevations for the bench marks. In the first adjustment, eight complete land circuits were used, the sea level connections at New York, Boston and Portland being ignored, as it has been found from an adjustment of all the first-order leveling in the United States that there is a decided slope to the mean sea level surface and that the closures of circuits of leveling involving connections with sea level are, on an average, larger than those of complete land circuits. The average closure of the eight complete land circuits was at the rate of 0.068 mm. per kilometer (0.0043 in. per mile), while the average closure of the two circuits involving the sea level connections was at the rate of 0.28 mm. per kilometer (0.0177 in. per mile).

All the corrections to the land circuits were due only to the inherent errors in leveling; therefore they represent a true indication of the accuracy of the work. The results of this adjustment, however, have not been used in any way in the assignment of elevations for the bench marks.

In the second adjustment the plane of mean sea level at New York, Boston and Portland was held fixed at zero elevation, making two additional circuits. The elevations derived from this second adjustment have been assigned to bench marks. These elevations are preliminary and will probably be changed by a small fraction of a foot when the New England net is combined and adjusted with the first-order level net of the United States. Any future adjustment, however, will have little or no effect upon the relative elevations of adjacent bench marks, nor should the relative elevations of bench marks several kilometers apart be affected appreciably.

Good Activated-Sludge Plant Effluent

Because a sample of sewage effluent from the activated-sludge plant of Pomona, Calif., lacked proper labeling and an opinion on its suitability for drinking and irrigation use was requested of the firm of chemists and engineers to which it was sent for analysis, the treated sewage was regarded as a sample of water and was reported on accordingly by the analysts, who attested its quality on the face value of the analysis but with qualifications as to sufficiently high nitrogen and suggestions as to the advisability of bacterial analyses. The effluent in question is used for irrigation. A portion of the report of the analyst follows:

From the standpoint of the mineral matter in solution in the sample this is a satisfactory water for drinking and other domestic purposes and for irrigation use. The presence of phosphoric anhydride in the amount of 0.55 grain per gallon and potassium oxide in the amount of 0.29 grain per gallon indicates that the water would be more beneficial for irrigation purposes than average water in this district. The nitrogen content of 1.27 grains per gallon is very high, and this would be of value in irrigation. It indicates, however, that the water may have been polluted by sewage and may not be satisfactory for drinking. To determine whether the water is potable, bacteriological examinations of it should be made and an examination of the source of the water should also be made to determine whether it would be constant in quality from the standpoint of its use for drinking."

Heavy Frost Damage to Roads Follows Open Winter

States in Frost Zone Report Widely Varying Effects of Open Winter With Frequent Thaws, but General Damage to Traffic-Bound and Lightly Surfaced Types

THE past winter was notable over a wide area as a winter of little snow and frequent thaws following a fall of generally heavy rains. These conditions promised severe frost action on highways. In most states the survey of damages is not yet completed, but a preliminary accounting is available, and it indicates widespread frost damage except in a few states where the cold was not severe or other ameliorating conditions prevailed. As reported by wire and letter in response to telegraphic inquiry, the situation as well as it can be determined at this date is as follows:

Maine, L. D. Barrows, State Highway Engineer—It is almost impossible to give at this time an intelligent estimate of the damage to highways by frost. In the northern part of the state there is still snow in places and the frost has not yet begun to act. The unimproved roads and some of the older gravel roads are in very bad shape, but we have not had much difficulty with hard-surfaced roads, such as bituminous macadam or concrete, especially the roads built during the last four or five years. Last fall we had a large amount of rain before freezing weather and have had very little snow this winter, especially in the southern part of the state. About ten days ago we had four or five days of warm weather, and this was just enough to start the frost in good shape. A. J. Wiggin, superintendent of maintenance, estimates that it will cost $500,000 to put the roads in satisfactory condition.

Massachusetts, A. W. Dean, State Highway Engineer—The expenditures in correcting frost breaks on state highways will not exceed $50,000 and may not reach that figure. We experienced intermittent frost breaks on about 500 miles of state highway, but this is not uncommon on the old state highways; in fact, the frost action this year is not much worse than it has been in previous years. There was perhaps a little more damage than in previous years, as the roads, being open, tempted people to travel more than they otherwise would, the travel extending over some roads that in past years have not been cleared of snow for traffic.

Connecticut, John A. Macdonald, State Highway Commissioner—The situation in Connecticut involves principally the old-type highways, such as the gravel highway and the light macadam highway. I should imagine that, on these, about 600 miles have received pretty bad treatment from the frost, and it will probably cost us $75,000 to put them back in shape. There is no particular kind of subsoil involved; the damage is mostly on the older types which are receiving considerable heavy traffic.

Rhode Island, G. H. Henderson, State Highway Engineer—Rhode Island state highways, after a winter of excessive frost action, are, with a very few exceptions, in excellent condition. As a matter of fact, we confined our annual weight restrictions this year to twenty roads, whereas last year they prevailed over our entire system, comprising upward of 80 roads. The only sections of state road that broke up in the spring were some very old waterbound macadams originally built in 1906 and 1907 and which have broken up continually in the past and will continue to break up until the subgrade is stabilized by reconstruction. At an outside figure, I would say that we have not more than 10 miles in this class. Most of our state roads are hard-surfaced, and on all roads constructed since 1913 the subgrade was stabilized by the installation of adequate drainage and gravel or stone foundations under the pavement. We are blessed with a large amount of coarse, clean gravel, which we have found especially adapted to foundation purposes. Through heavy retentive soils we generally construct a 9- or 12-in. layer under the pavement, while in less retentive soils a 6-in. layer will generally suffice. In sections where the natural soil is gravelly or permeable, of course, we have not found foundations necessary. The low-type town roads throughout the state doubtless were in the worst condition this spring that they have been in for several years, and many of the roads which have been fair in other years at this time of the year were almost impassable this year.

New York, A. W. Brandt, Commissioner of Highways—It is the same old condition this year that we have had in past years except that it is much more alarming; a greater mileage of the old thin macadam roads is broken up this year than has ever been the case before. Just to what to ascribe it, I don't know. I have given up prophesying what certain types of winter will do to our roads, but it is true that, while we had a wonderful fall last year for work, we had a great deal of rain in the latter part of November and early in December which left the ground in a most saturated condition. This, with the absence of snow, which we have always considered as a protection, and probably a greater amount of heavy trucking at the critical period, is the reason for the exceptionally broken-up condition.

New Jersey, W. G. Sloan, State Highway Engineer—So far as the state highway system of New Jersey is concerned, there is no report to make of excessive damage on account of frost action. As a matter of fact, the maintenance division advises that typical frost damage has been practically entirely absent this year. The absence of such damage is due, in our opinion, in a great measure at least, to the fact that weather conditions have been such in New Jersey during the past winter that the frost has not penetrated the ground to any considerable depth. Reports indicate that even in the extreme northerly end of the state the frost penetration out on the unpaved and ungraded sides of the highway did not exceed 1 ft.

Ohio, Harry J. Kirk, State Highway Engineer—Any estimate in reference to frost damage is of necessity very much of a wild guess. It is, however, possible to compare the damage done one season with that in another season. Our experience is that the winter of 1926-27 was characterized by a very great frost damage. This we attribute to two conditions:

1. A very wet fall, so that our highways entered the winter with the soil completely saturated with moisture.

2. Repeated freezing and thawing beyond the ordinary.

Every year we take pains to try to relieve frost conditions by various means, particularly drainage, so that as time goes on we overcome much of the frost damage. I do not consider the frost damage during the past winter as much more than half what it was during the winter of

1926-27. I judge this from different sections of road that have gone through the winter in much better condition than they did the previous winter. Emphasis must be made of the difficulty of separating frost damage from other causes of road deterioration. This condition makes it very difficult to get anything but a guess on the total amount of this damage.

Indiana, A. H. Hinkle, Superintendent of Maintenance—This last winter has been an unusually bad winter on our highways, particularly the gravel and stone roads. The unusual damage to these roads from frost action was undoubtedly largely due to the fact that we had three thaws during the middle of winter—one in the latter part of December, one in January and another in the early part of February. Naturally when the frost comes out of the gravel or stone road in the early winter, it leaves the metal loose and open, permitting greater absorption of water than under normal conditions, and this excess of water, again freezing, causes greater destruction to the road surface. While in the southern part of the state we have these thaws frequently during the winter season, they come more frequently this winter than usual, particularly in the central and north-central parts of the state.

Although we had these excessive winter conditions which were destructive to our highways, yet due to the heavier coat of metal on most of our gravel and stone roads, to more systematic maintenance and to a better regulation of heavy trucking during the thawing season, I think I am safe in saying that as a whole the soft condition of our roads did not handicap automobile traffic much more this season than in the average season in the past. It would be difficult or almost impossible for me to estimate the additional cost incurred by these winter thaws over the usual conditions of preceding years. The latter part of the winter and early spring was comparatively free from heavy rains and the roads dried out about as fast as they thawed. I therefore doubt whether we shall pay very much more for winter and spring maintenance combined this season than we have paid on an average for the past seven years. With an up-to-date, live maintenance organization that drags and scrapes the road intensively while it is thawing, the ruts and breaks that are produced soon heal up, particularly if we happen to have a short period of good weather.

Wisconsin, H. J. Kuelling, State Highway Engineer—The so-called "frost boils" or "frost damage" this spring in Wisconsin was materially less than last year. Naturally we have some impassable places on our gravel or stone surfaced roads, not to mention the earth roads. These places are short and not very expensive to repair, although they are sufficient to hold up traffic. The actual cost of repairs on gravel roads is about $50,000, although this is merely a wild guess. In so far as our concrete roads are concerned, the frost damage has been quite negligible, and in no cases that I know of has there been any delay to traffic.

Minnesota, J. T. Ellison, State Highway Engineer—The damage to our highways from frost action cannot be determined at this time, as it is not possible to estimate the recovery until after the spring break-up is over. I believe that the loss to interrupted traffic and delayed business is probably much greater than the direct damage to the roads, although there is no question that the damage to our highways is very serious. About 2,000 miles of our gravel roads is affected at the present time, but this mileage may be considerably increased when the frost comes out farther north in the state. At the present time our difficulty is confined largely to the southern half of the state, but it is entirely possible that the situation will become still worse and that we shall encounter serious trouble in the northern portions of the state where as yet no serious breaks have occurred. Our situation is not as bad as it was a year ago, due largely, I believe, to prompt load restrictions being imposed as soon as signs of trouble appeared, and a strict enforcement of the restrictions. Representatives of the Bureau of Public Roads are now here for the purpose of making a study of soils and frost action, and we are co-operating with them.

Michigan, G. C. Dillman, State Highway Engineer—The estimated damage to roads in Michigan due to excessive frost action this spring amounts to the following approximate figures: On 320 miles of gravel, $50 per mile; on 570 miles of macadam, $100 per mile; on 2,000 miles of pavement, $150 per mile.

Oregon, R. H. Baldock, Maintenance Engineer—As stated last week, the state roads suffered damage amounting to $43,000 on 50 miles of our highways by reason of excessive frost action this spring. This occurred on two sections. One section, approximately 40 miles in length, was on the old Oregon Trail, U. S. Highway, 30, between Pendleton and La Grande through the Blue Mountains. This section of highway traverses a mountain region, with an elevation ranging from 3,000 to 4,200 ft. The soil is a heavy clay overlying basalt rock. The highway has been constructed approximately five years, and each spring we have suffered some frost damage, but there was particularly severe damage this winter for the following reasons:

The ground was supersaturated by heavy rains last fall and the frost went deeper than ever has been known before during our experience. About the middle of March we had an unusually warm spell, under which conditions the frost left the ground very rapidly. This combination of circumstances caused a severe break-up with resulting damage of approximately $1,000 per mile.

The other section is a portion of the same highway approximately 80 miles to the east of the former section and located in the Powder River Valley near the city of Baker. The highway at this point traverses a very flat country. The soil is a very fine silt, heavily charged with alkali. We have had trouble through this section each spring since the highway was built, but two years ago we materially strengthened the weaker portions by the addition of approximately 14 in. of crushed rock. The drainage through this section has been well cared for. At the low points the water in the ditches rises to about 30 in. below the crown of the road. The water does not stand in the ditch line, but has been carefully drained into various sloughs traversing this flat. Each fall an extreme capillary action is manifested and the roadbed becomes saturated with water, and is later frozen solid, in which condition it remains through the entire winter.

In many places frost boils become evident, causing rolls in the oiled macadam surfacing from 3 to 8 in. high and from 2 to 10 ft. in length. Formerly we were able to remove these frost boils by use of steam in accordance with the method that appeared in the *Engineering News-Record* some three years ago, but this last winter the frost was so deep that we were unable to use this method. The conditions last winter on this section were more severe than has been evident during our experience. However, the resurfacing with additional materials prevented the break-up in all but a few instances. The damage on this 10 miles was about $3,000.

From Job and Office

Hints That Cut Cost and Time for the Contractor and Engineer

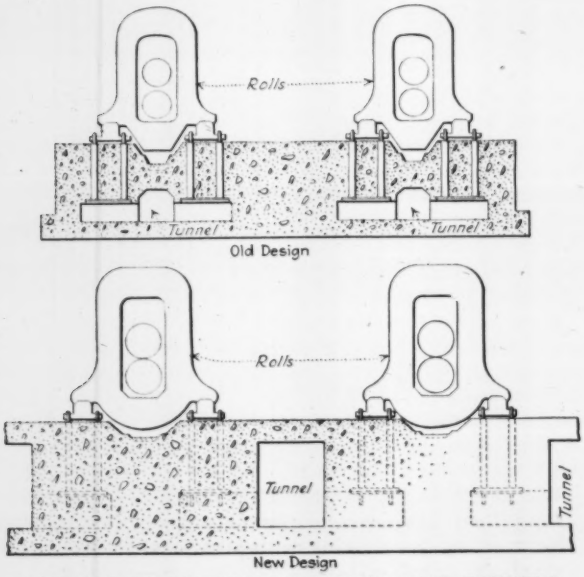
Trailer Permits Use of Trencher on Small Widely Separated Jobs

BY carrying a small-sized trench excavator on a specially designed 15-ton trailer, the Springfield (Mass.) water department has been able to move the excavator about to small jobs which were previously excavated by hand.

The trailer consists of a platform 8x20 ft. in size, mounted 20 in. from the ground, on solid rubber-tired wheels. The trailer is coupled to the rear of a truck and may be moved at a speed of 20 miles per hour. Under its own power, equipped with a crawler, it was possible to move the trailer at a speed of only 8 miles per hour. For this reason it was previously used only on the larger jobs or on a number of small jobs located near to each other.

New Design for Anchor Bolt Tunnel in Mill Settings

THE foundations for roll installations in steel mills usually incorporate tunnels giving access to the anchor bolts. In the new mill at the Middletown (Ohio) plant of the American Rolling Mill Company a number of advantages are believed to be gained by a new design for these foundations. In the accompanying illustration are shown both the old and the new design. The old design involved the use of a tunnel for the bolt pockets directly under each set of rolls. In the new design such tunnels are located between the rolls, one tunnel giving access to two adjacent rolls. In the old arrangement the tunnels had to be very shallow, whereas the new tunnels can be made high enough to permit a man to walk through them. All such tunnels at best are dirty on account of water running down through the bolt sleeves, and with the old arrangement it was almost impossible to clean them out. With the new arrangement, cleaning will be made easier and will therefore be done more often.



OLD AND NEW DESIGN OF ANCHOR BOLT TUNNELS IN ROLLING MILL FOUNDATIONS

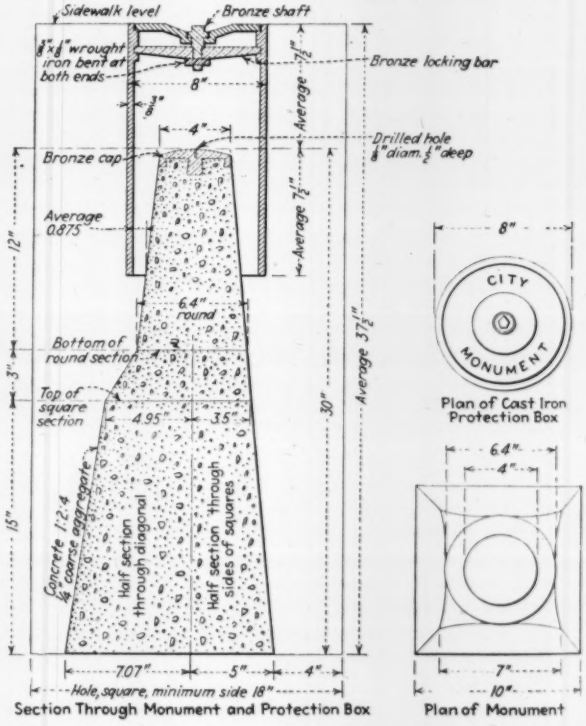
Topographical Survey Monuments Used by Reading, Pa.

By E. Clinton Weber
City Engineer, Reading, Pa.

THE accompanying sketch shows the detailed construction of the topographical survey monument recently adopted by the city of Reading, Pa., to mark street alignments and bench mark elevations. The principal features connected with this design are as follows:

1. The base is 10 in. square, presenting a bedding surface more than six times that heretofore provided by the 4-in. square base of the ordinary street monument.

2. The depth of the monument, coupled with its tapering sides, will prevent its being lifted by the action of frost.



DETAILS OF RECENTLY ADOPTED SURVEY MONUMENT

3. The monument is made square in cross-section throughout its bottom half to permit the wedging of stones all around between the sides of the monument and the sides of the hole dug for its reception. The bottom half of the monument is a frustum of a square pyramid, tapering from a side of 10 in. at the bottom to 7 in. at the top.

4. The top section of the monument is a frustum of

a cone tapering from a diameter of 6.4 in. at the bottom to 4 in. at the top for a height of 12 in. This shape was adopted to permit the protection box to be raised, lowered or turned without disturbing the monument.

5. The mid-section of the monument is 3 in. in height and changes gradually from a circular to a square cross-section.

6. Four deformed steel reinforcing bars, not shown on the drawing, are inserted throughout the depth of the monument.

7. The space between the sides of the hole dug for the monument's reception and the cast-iron protection box and monument, not occupied by the wedging stones, is filled with boiler cinders, thoroughly and evenly tamped into position up to the sidewalk paving or to the proposed sidewalk elevation.

8. The monument and protection box when first placed in position are set in the mean position shown on the plans, thus permitting the lowering or raising of the sidewalk level 4½ in. either way without disturbing the monument.

9. The protection box is flexible enough to permit its tilting to conform to the slope of the sidewalk.

10. The top of the monument is made of tobin bronze with a hole drilled into its center for the purpose of marking the alignment. The top of the bronze cap is convex in shape so as to present but one point, the highest, to the bottom of the leveling rod placed thereon when running levels. The elevation of the monument when once established is not likely to be changed by means of abrasion. Old-time monuments are worn down by traffic and portions are frequently chipped off the tops and sides, and in many instances entire tops are completely broken off.

11. No lateral disturbance is likely to displace the monument, because the protection box will usually take care of any disturbance of this kind.

The monuments and protection boxes were made by the Reading Meter Repair Company, Reading, Pa.

Striping California Highways to Mark Traffic Lanes

STRIPES on paved highways have been found to be so effective as a safety measure in California that the plan is being widely used on the highway system in that state, both on narrow, winding roads in mountainous country and on the four-lane pavements of trunk routes where tangents are long. On the curves of the mountainous routes the stripe helps because it reminds the driver to keep on the right side of the center line, and on wide straightaways in level country the demarcation of the several traffic lanes helps to keep slow-moving vehicles in the outer lanes, leaving the inner lanes for high-speed traffic. In fog belts and on desert pavement with oiled shoulders, where it is difficult to distinguish the edge of the pavement at night, the guide line is a great help to drivers; particularly does it permit of freer passing by fast machines, thus increasing the safe average speed.

TYPICAL USE OF GUIDE STRIPE ON MOUNTAIN PAVEMENT

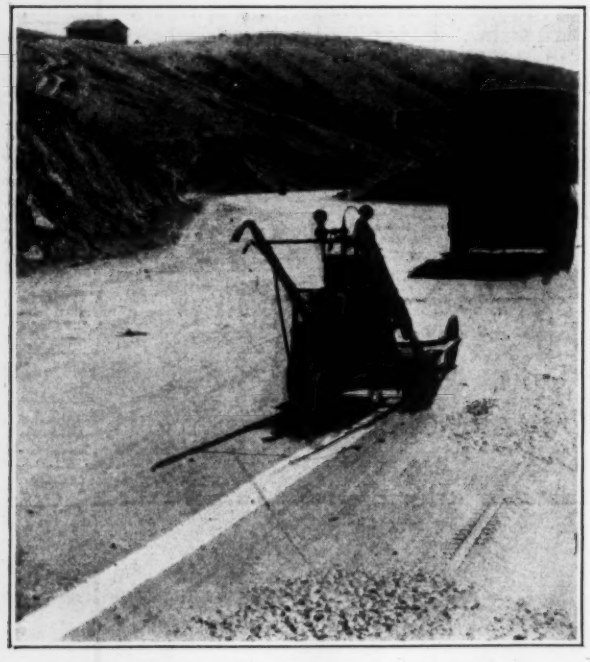
Traffic lanes were formerly marked in California only at intersections, on blind curves and on narrow bridges. The result was so satisfactory that the use of the stripe was extended to long stretches of heavily traveled routes. To date 260 miles of state highways have been striped and the mileage is steadily increasing.

Because slight variations in the alignment are magnified to a remarkable degree when seen from the viewpoint of the automobile driver, careful attention to the striping operations is required. Particularly on rough pavement with ragged edges it is very difficult to make a truly uniform stripe. For pavements with a uniform edge a wheel device with a marker arm at the center of the pavement may be used to strike a line for the painting machine to follow. On other pavements it has been found necessary to mark the line by means of a cable the position of which is determined by measurement.

Important requirements are that the paint must work rapidly through the applying mechanism, spread uni-

formly, dry quickly without becoming chalky and wear well under traffic. At first a white paint was used, but the preference now is for an orange-colored traffic lacquer paint. For a single 6-in. stripe, which is the standard width, the amount of paint required varies from 10 to 15 gal. per stripe-mile. Under present methods of application the cost varies from about $25 to $55 per mile, with an average of about $38. The variation in quantity of paint applied indicates an uneven distribution with probable inequality in rate of wear and life.

The machine now used most extensively by the division of highways sprays the paint on the pavement with the



REAR VIEW OF STRIPING MACHINE AND ATTENDING TRUCK

aid of compressed air. Two plates set vertically 6 in. apart retain the spray within the prescribed width limit, thus keeping the edges of the stripe sharp and distinct. In this machine a guide wheel about 5 ft. ahead of the point of application keeps the machine accurately to the mark previously placed on the pavement, thus giving considerable leverage, as it were, in keeping the stripe on the desired alignment.

The device is built on a light steel frame carried by rubber-tired wheels. On this frame is mounted an air compressor driven by a small gasoline motor and a 20-gal. paint tank. The device is equipped with wooden handles, very similar to plow handles, so that the operator can use both hands in pushing it along and guiding it to the prescribed alignment. The truck in which supplies and equipment are moved is driven slowly ahead of the striping machine to safeguard the machine operator from oncoming traffic.

Although a very quick drying paint is used, it is customary to place "keep-off" markers or large stones on the newly painted stripes, these being removed by the truck crew at the end of the day's work. The paint now used dries so that it will bear traffic without damage in 45 minutes to two hours, according to weather conditions. Commercial grades of traffic lacquer paint are used, no stated specifications being employed in their selection.

From Job and Office

Hints That Cut Cost and Time

Various machines have been used for marking the stripes on the pavement; the Division of Highways has developed some of its own and the one shown in the accompanying picture is a patented device made by the Simmons Company. Striping and stripe maintenance are done by the maintenance department of the Division of Highways, to which *Engineering News-Record* is indebted for the picture and data in this note.

Subway Work Requires Heavy Drilling Through Retaining Walls

IN THE construction of the Eighth Ave. subway, New York City, extreme care was necessary in excavating between the General Post Office and the Pennsylvania Station. This subway crosses over the numerous tracks of the Pennsylvania and Long Island railroads. Retaining walls 11 ft. thick and 19 ft. high had been constructed to protect the foundation walls of the General Post Office and Pennsylvania Station, and though designed to allow for a future subway, insufficient room had been left for sewers, water mains and cable ducts. This made necessary the removal of the major portions of the wall.

In many places these walls were reduced from the original 11-ft. thickness to 1 ft., and the possibility of cracking the remainder of the wall was always imminent. For that reason all of the wall removal was by channeling with air drills, it being inadvisable to use explosives to any great extent in so delicate an operation. The additional fact that these walls extend over the entire station platform made it essential that extreme caution be used so as not to endanger anyone in the station below.

With this idea in mind a survey was made of the situation and the decision of the contractor and his engineers was that successive portions of the wall should be partly detached by line drilling and demolition then carried on by the plug and feather method. Selection of a proper rock drill was necessary, for the drilling had to be done

LINE DRILLING ON HEAVY RETAINING WALL

From Job and Office

For Contractor and Engineer

not only through rock and concrete, but through heavy layers of waterproofing. It was also necessary to drill a line of deep holes, each in the same plane and accurately aligned.

As holes varied from 2 to 18 ft., with the vertical holes generally the longer, a special chuck end was developed that permitted the use of 1-in. hollow hexagon-collared drill steel in a machine intended for use with 1¼-in. hollow round steel. Practically all of the drilling was done from quarry bars, almost at street level.

In the early stages of the work the vertical holes were drilled on 3-in. centers and the horizontal holes on 2½-in. centers. This latter line was then broached, after which plugs and feathers were used to complete the removal. This method soon proved too slow and it was decided to try blasting. A skillful blasting foreman was selected for this work and careful supervision was given by the engineering department of the contracting firm. Two weeks before blasting was done dowel holes were drilled in the portion of the wall that was to be left standing. These were drilled on 3-ft. centers to a depth of 4 ft. below the removal line. Into each hole a 1-in. square steel rod was grouted. The first line of holes for blasting was drilled 20 in. back and at 6-in. centers. Every fourth hole was charged with 2 in. of 40 per cent dynamite. Five charges were fired at a time. As the wall became thinner the successive lifts were reduced until the last one was but 10 in. thick. Drill holes were spaced closer on successive lifts until on the last one they were being drilled on 2-in. centers.

The work was carried out under the engineering supervision of C. N. Ely, with John Holden as construction superintendent for the contractor, the Rosoff Subway Construction Company. W. A. Murphy was the field engineer on the job. The information contained in this article was supplied by the Denver Rock Drill Manufacturing Company (now the Gardner-Denver Company), whose drills were used on this work.

Paving Mixer Speeds Up Curb Pouring in Detroit, Mich.

THE USE of a 21-E paving mixer in pouring continuous curb and combined curb and gutter is a radical change from the ordinary methods of wheelbarrow handling at both ends of the mixer. This system, employed last year in Detroit, Mich., has produced daily average pours of 2,500 lin.ft., with a maximum of 4,063 lin. ft. for curb alone and of 2,800 lin.ft. for combined curb and gutter. The former small mixer and wheelbarrow crew of 26 to 28 men usually poured 500 lin.ft. in one day. A paver and a crew of 37 men are required by the new method.

The forms are built after the rough grading is completed, but before fine grading. The paver, equipped with a full caterpillar tread and an old-style delivery spout, is run alongside at a distance of 4 ft. from the forms. A gate near the end of the spout controls the concrete flow into the forms. Two concrete spaders follow close behind the spout and are in turn followed by two float men, one bullnose man, two finishers and one brush man, in the order named. The aggregates are delivered to the mixer by trucks from near-by stockpiles.

PAVING MIXER POURING CONCRETE CURB

The finished curb section is 20 in. deep and 6 in. wide, divided by steel templets into 6-ft. blocks. These templets are removed by the last man in the finishing crew. To take care of expansion, a ⅜-in. asphalt strip joint was placed every four blocks, or 24 ft. A carefully proportioned 1:2:3 mix was used, held in the mixer a full minute. By using carefully graded aggregate of 1-in. maximum size and the correct amount of mixing water and by proper spading of the concrete after placing, an excellent curb was always obtained.

This type of curb has been removed and set back to permit street widening, which is a distinct advantage to a rapidly growing community.

Job and Office Notes

RICE HULLS ARE BEING USED by California contractors to dust the interiors of motor trucks in handling wet or clayey material. Grading operations conducted in winter months have caused these contractors considerable expense and loss of time in handling wet materials. Certain soils and clays are more or less tenacious upon absorbing a small quantity of water, says a recent issue of California *Highways and Public Works*, and, although passing through the bucket on the power excavating equipment without great difficulty, once loaded into trucks or other hauling equipment and transported any distance the load often has to be re-excavated by hand before it will discharge. A light covering of rice hulls has been found sufficient to start the load and it discharges readily when the truck bed is raised. These hulls are a waste product at all California rice mills and on the particular job herein mentioned, that of the Ariss-Knapp Company's job in Alameda County, the haul for these rice hulls was about 30 miles.

Letters to the Editor

A Forum for Discussion of Views of Engineers and Contractors

Dam Failure by Sliding a Century Ago

Sir—I beg to congratulate you on the splendid preliminary report of the failure of the St. Francis dam in your issue of March 22. I wish to call to your attention the failure of a very similar dam in Spain 120 years ago.

In P. Ziegler's "Thalsperrenbau" (Berlin 1911, Wm. Ernst & Sohn) is described the failure of the Puentes dam in Spain, built in 1785-91, 167 ft. high, 152 ft. thick at the base and about 900 ft. long, somewhat curved in plan. The dam was not founded on bedrock for 67 ft., and when the water reached the level of 155 ft. the pile foundation was pushed out like a cork from a bottle and a part of the rubble masonry of the center part of the dam 67 ft. wide and 100 ft. deep was carried away. The entire dam was not destroyed, because the dam acted as an arch and found good abutments in the dense limestone canyon walls.

Evidently in the St. Francis dam both foundation and one side of the bank gave way at the same time, thus not allowing the thick dam to act as an arch, with the result of the destruction of nearly the entire dam.

L. J. MENSCH.

Chicago, March 31, 1928.

Sludge Disposal at the Milwaukee Activated-Sludge Plant

Sir—Referring to the editorial on sewage-sludge disposal entitled "Still Unsolved," published in your issue of Jan. 26, 1928, p. 138, the writer questions the title and the inference to be drawn from the statement in the first paragraph, "but at the comparatively few activated-sludge plants where attempts have been made to dewater the sludge for the sake of its high nitrogen content there has been no end of trouble."

The desire of able engineers to solve the problem of disposal of the sludge produced by the activated-sludge process has brought about many unsuccessful methods, due largely to their aim for low capital and operating costs. So far as the writer knows, no such method has been found which he would care to adopt. No doubt the history of these various methods has caused the statement quoted above.

The writer believes that, for the larger installations of the activated-sludge process, the successful disposal of the sludge has been solved at the Milwaukee plant. This has not been done without overcoming the mechanical troubles usually present in starting up any industrial plant dealing with a new product, but the troubles have been largely mechanical and not chemical, and in judging the history of the Milwaukee plant it must be remembered that it is the pioneer plant of any considerable magnitude where the complete process was to be carried out, and all designs were made to conform with the results obtained from the laboratory and small demonstration plants, utilizing, of course, the results obtained by other researchers.

The chief difficulties were the selection of the proper type of pumps to handle the sludge, the proper type of filter cloth, conveyors and driers. All these difficulties have been successfully oversome so that today the plant is handling the full amount of sewage for which it was designed, producing a higher standard of effluent than that adopted by the sewerage commission and 100 tons of dried sludge per day, which was the original estimate.

So far as the conditioner is concerned, sulphuric acid was first used; then aluminum sulphate was added to meet certain conditions of the sludge; early in 1927 ferric chloride was added to meet the most adverse conditions. So far, throughout the winter months, ferric chloride has been very little used, dependence being placed almost entirely upon the aluminum sulphate. As the sewage warms up, sulphuric acid will be used.

In spite of the unfortunate adverse criticism of the Milwaukee plant, which has been pretty generally circulated, it is doing all or even more than its designers expected and has successfully solved the sludge problem. Within one year it has increased the amount of sludge produced from 40 to 107 tons per day. It has disposed of all of the sludge produced at a price exceeding the cost of production and has built up a market that cannot possibly be met by the maximum volume produced.

In 1927, when the sludge production amounted to approximately 20,000 tons, the income from the sale of sludge was between $300,000 and $400,000. It is estimated by the chief engineer, Robert Cramer, that the return from the sale of the sludge during 1928 will be between $550,000 and $600,000 and that the operating expenses will approximate $900,000, making the total cost of sewage disposal to the present metropolitan district between 60c. and 65c. per capita.

It is the writer's belief, after a good many years' study of the activated-sludge process, that plants of a considerable magnitude cannot be economically operated unless a considerable part of the operating costs can be met by the sale of the sludge produced and that Milwaukee has solved the problem of successfully disposing of the sludge.

The writer wishes to call attention to the papers entitled "Scientific Sewage Disposal at Milwaukee," by Robert Cramer and John Arthur Wilson, and "Milorganite—A New Fertilizer Material," by Victor H. Kadish, appearing in the January issue of *Industrial and Engineering Chemistry*.

Milwaukee, Wis., Feb. 29, 1928.

T. CHALKLEY HATTON, Consulting Engineer.

[From the article by Mr. Kadish, mentioned by Mr. Hatton, it appears that "Milorgarnite production for the calendar year 1926, during a large part of which operations were intermittent, totaled about 5,000 tons, of which 2,200 tons were shipped for sale, and that for the first 10½ months of 1927 the production was 16,600 tons and the shipments for sale 19,700 tons." The "average return," Mr. Kadish states, "has been about $15 per ton f.o.b. Milwaukee. Booked orders up to May 1, 1928, total 13,000, or the estimated production to that date." At the close of 1927, states Mr. Kadish, "with an average of 80 m.g.d. of sewage treated daily, the monthly production of Milorganite is about 2,500 tons. For the year 1928 it is estimated the output will run between 30,000 and 35,000 tons." The successful marketing of Milorganite is no longer problematical and every ton produced could be placed.

In the article by Mr. Cramer, mentioned by Mr. Hatton, the cost of the Milwaukee sewage-works is given as about $10,000,000 and the returns from the sale of fertilizer about $500,000 per year.

Whether the $10,000,000 given as the cost of the sewage-works includes interest during construction does not appear, but if interest is included, then Milwaukee is an exception to the general practice of municipalities in reporting the capital cost of their various enterprises.—EDITOR.]

Tensile Rivet Stresses

Sir—In the discussion by H. E. Wessman in your issue of April 12, p. 602, on tensile working stress for rivets, Professor Wessman's argument that the amount of stress taken by the rivet depends upon the proportion of total strain in the plate and rivet respectively is perfectly logical.

The writer would point out, however, that the rivet which is in tension is perfectly free to contract laterally, whereas the plate in compression has lateral restraint, which decreases enormously the strain effect on the plate caused by a given load.

While theoretically there would be a slight accretion to the stress in the rivet when a load is placed on it, this would be such a small percentage that it could safely be neglected. Therefore the generally understood theorem, that a rivet under initial stress receives no increase in stress until the external load imposed is large enough to exceed this initial tension, can be considered correct for all practical purposes.

Philadelphia, Pa., April 14, 1928.

A. S. WOODLE, JR., Engineer of Plant, Baldwin Locomotive Works.

News of the Week

Current Events in the Civil Engineering and Contracting Fields

Mead Committee Is in Agreement With Others On St. Francis Failure

Failure Due to Character of Rock and Not to Earth Movement or Faulty Design

SUBSTANTIAL agreement with the other investigators is found in the report of the committee headed by Elwood Mead, chief of the Reclamation Service of the U. S. Department of the Interior, a committee composed of, in addition to Mr. Mead, Louis C. Hill and Lansing H. Beach, with D. C. Henny and R. F. Walter as consultants.

The committee concluded that: (1) The type and dimensions of the dam were amply sufficient if based on suitable foundation; (2) the concrete of which the dam was built was of ample strength to resist the stresses to which it would be normally subjected; (3) the failure cannot be laid to movements of the earth's crust; (4) the dam failed as a result of defective foundations; (5) this failure reflects in no way on the stability of a well-designed gravity dam properly founded on suitable bedrock.

The character of the rock as determined by the committee is much as has been described in earlier reports, particularly that of Mr. Bowers in our issue of March 22. A fact which has not heretofore been brought out was a statement made to the committee by the superintendent of construction of the dam to the effect that the rock in the stream bed was excavated with a steam shovel, picks and bars. The cutoff trench was not continuous under the whole foundation and was excavated with picks and bars, in no place to a width or depth of over 3 ft.

Seepage Below Dam

Referring to the seepage of water below the dam, the committee states that the character of the rock was such that water flowing through it could have dissolved out much of the gypsum and so continue to be clear, while at the same time progressively weakening the rock. The muddiness of the water was due, according to Mr. Mulholland, to the fact that grading was being done on a road above the point where water was coming through the rock and that some of the excavated material was dumped into the streams of water running down the side hills, causing them to become muddy. He was positive in his statement to the effect that no muddy water was issuing from under the dam.

Examinations of the records of the Water Department showed that the reservoir never had been filled until eight days before the failure, although it had been nearly full for a couple of weeks in May of last year.

A more complete abstract of the report will be given in a subsequent issue.

Pennsylvania R. R. Announces Plans for Newark Terminal

Pending the result of negotiations between the Pennsylvania Railroad, the city of Newark, N. J., the New Jersey State Highway Department and the Public Service Corporation, announcement has been made of tentative plans for the proposed improvements in transportation facilities at Newark.

Besides the replacement of the present Market St. station of the Pennsylvania, there will be a new trolley subway along the bed of the old Morris Canal, new streets to care for increased traffic and a plaza facing the station. Provision will be made for the transfer of through passengers from Pennsylvania trains to the rapid-transit service of the Hudson & Manhattan tubes as well as for local service on both lines.

The proposed plans call for six tracks for Pennsylvania Railroad traffic, with two additional for Hudson & Manhattan trains. All of these will be at the present track level except one, which will be elevated 18 ft. This latter will be the unloading track for rapid-transit tube trains, and will connect with both the street and a lower platform to provide for local and through passengers.

The main concourse under the tracks will contain all usual facilities and conveniences. A motor-bus terminal is to be established in connection with the new station. The city of Newark and the Public Service Corporation are planning for adequate trolley and bus service and the opening of new streets to care for the increased traffic. A new boulevard on the north side of the tracks, extending from Pioneer St. to Front St., will be a part of the state highway system.

When completed, the new station will care for all through and local traffic on both the Pennsylvania Railroad and the Hudson & Manhattan tubes. The present station on the meadows, known as Manhattan Transfer, will be abandoned.

Annual Meeting of Wood Utilization Committee

The annual meeting of the National Committee on Wood Utilization of the Department of Commerce has been called for May 4 and 5, in the Commerce Building, Washington, D. C. This meeting will follow a lumber conference called by Secretary Hoover, May 3 and 4.

Civil Engineers Ask Further Study of Flood Control on Mississippi

Resolution Presented to President and Congressmen—Board Acts on St. Francis Dam

MISSISSIPPI flood-control legislation was the subject of resolutions adopted by the board of direction of the American Society of Civil Engineers, meeting in Washington on April 23.

Whereas the control of Mississippi River floods affects important interests other than navigation, including agriculture, railway and highway transportation, the operation of industry and business, and the water supply and sanitation of municipalities; and

Whereas many features of the proposed control and regulating works will be unprecedented in design and magnitude and will cost much more than any works heretofore undertaken by the federal government; and

Whereas no representative commercial or industrial organization would undertake a task of comparable nature without the most thorough review and analysis of the character, details and scope of the proposed plans by competent agencies independent of the personnel selected for the accomplishment of the task; now therefore be it

Resolved, That the board of direction of the American Society of Civil Engineers:

First: Commends the President for his position in insisting upon the very important principle of local participation in the cost of protection in proportion to the benefits derived and in proportion to the resources and responsibilities of the interests affected;

Second: Approves the principle of placing the responsibility for construction in an executive department;

Third: Inasmuch as a great amount of flood-control work on the Mississippi River remains unfinished which already has been approved by the Congress or is generally agreed upon as essential to any comprehensive plan, that the Congress be requested to make appropriations without delay for such work, to be constructed at once without waiting for the completion of a general plan;

Fourth: Earnestly recommends the creation of a board of at least nine disinterested engineers, chosen by the President for their outstanding training, experience and accomplishments in hydraulic and river-control problems, and in the design, organization and construction of large and important public works, this board to confer with the departmental officials responsible for the preparation of plans and to make a complete review and analysis and a report to the Congress on the scope and the technical and economic features of the comprehensive plan and program;

Fifth: In view of the paucity of data, incompleteness of surveys and lack of information necessary for solving many controversial features of the Mississippi River problems, that Congress be requested to authorize and appropriate sufficient funds for such studies and the creation of such a board to carry on such work and to report to the next Congress.

A copy of the resolutions was presented to President Coolidge by a committee composed of George S. Davison, past-president; Lincoln Bush, president; Arthur E. Morgan, vice-president; Morris Knowles, director, and George W. Fuller, vice-president of the society.

Copies of it were presented also to Charles G. Dawes, Vice-President; Nicholas Longworth, Speaker of the

House; Wesley L. Jones, chairman, Senate committee on commerce; Frank R. Reid, chairman, flood-control committee of the House; Gen. Edgar Jadwin, Chief of Engineers, and Dwight F. Davis, Secretary of War.

Twenty out of the 25 members of the board of direction of the American Society of Civil Engineers attended the meeting of the board held in Washington on April 23 and 24, immediately preceding the spring meeting of the society.

MEETING WELL ATTENDED

One member of the society was expelled by the board and three other cases were dealt with involving violations of the code of ethics. Approval was given by the board to answers prepared by the committee on professional conduct to two hypothetical questions on matters of professional practice.

A resolution was adopted recommending reorganization and concentration of the engineering functions of the federal government and favoring legislation designed to accomplish it. The executive committee was authorized to arrange for such representation before Congress on this subject as might seem advisable.

DATA ON ST. FRANCIS DAM

A committee of three was authorized to prepare in concise form as a matter of record a codification of the essential facts brought out in the several reports on the failure of the St. Francis dam. The board authorized also the appointment of a committee of two to urge in appropriate legislative quarters that jurisdiction over the proposed governmental hydraulic laboratory be placed with the Bureau of Standards.

Bylaws were adopted providing that applicants for membership in the society resident outside the United States and Canada shall have the approval of two corporate members and that they shall be members of some recognized engineering society in their own country, if there be any such.

The board authorized the appointment of a committee to consist of Harrison P. Eddy and James H. Johnston to confer with the judiciary committees of Congress with a view to working out some practicable program to relieve the dependents of the engineers who lost their lives in the "Norman" disaster on the Mississippi River in May, 1925.

The board accepted with appreciation the gift of Charles Evan Fowler amounting to $10,000 to establish a Phebe Hobson Fowler fund, the income from which is to provide awards for conspicuous achievement in advancing the standing of the profession and to promote architectural excellence in the design of engineering structures.

A committee was authorized to prepare a list of engineering schools of approved standing for use in the membership work of the board and in considering applications from student chapters. Application for a student chapter at the University of New Hampshire was approved.

Major Sultan to Attend Navigation Congress

Major Daniel I. Sultan, Corps of Engineers, U. S. Army, has been ordered to proceed to Brussels and Ghent, Belgium, to attend the annual meeting of the Permanent International Commission of Navigation Congresses beginning May 14, 1928. Major Sultan, who is ex officio a member of the permanent commission of the association, has been designated as a representative of the American section, on account of the large interests of the United States in the improvement of navigation, and consequently, in the proceedings of the International Congresses of Navigation.

D. L. & W. Plans Electrification of Suburban Lines

Plans for the electrification of the suburban lines of the Delaware, Lackawanna & Western Railroad are in preparation, according to an announcement by J. M. Davis, president of the road. The work, which will cost from $14,000,000 to $18,000,000, has already been authorized and will be completed within two years.

The proposed electrification affects the Morris & Essex branch from Hoboken to Dover, the Passaic & Delaware branch from Hoboken to Gladstone and the Montclair branch, involving 76 miles of road, or 173 miles of track. The territory served has a population of about 1,250,000 persons.

The decision to proceed with the work was reached after a conference between President Davis and a joint committee representing business and civic organizations of 22 communities served by the railroad. The committee expressed its willingness to co-operate with the railroad in an effort to secure higher commutation rates in consideration of the better service expected.

Steamer Damages Drawbridge

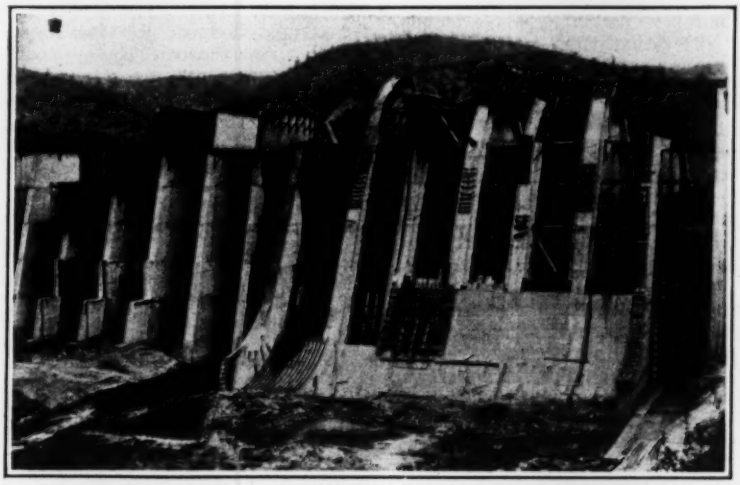
The Diversey Ave. swing bridge over the north branch of the Chicago River was struck by the sand dredge "Gilbert" on April 3, as the bridge was being opened. The span was displaced from the turntable so that it could not be operated and traffic was detoured until repairs could be made.

More Funds Needed to Complete Louvain Memorial

Additional funds are needed to insure the proper completion of the memorial carillon and clock tower of the new Louvain Library, which is being erected in memory of the American engineers who gave up their lives in the World War. Edward Dean Adams, chairman of the Engineering Foundation committee which is raising the fund, states that 2,659 donors have already contributed $62,112. Among the organizations which served in the World War and which have contributed as such are the 6th, 11th, 27th and 301st Engineers. A number of other regiments have the matter scheduled for early consideration.

The contracts for clock and carillon involve about $71,200. In addition, tablets, memorial volumes and other items mean an outlay of $1,800, and there will be a perpetual maintenance fund of $10,000, making a total cost of about $83,000. Work on the memorial is progressing satisfactorily, and dedication ceremonies are being planned for July 4.

The committee hopes that many of the 75,000 engineers eligible to contribute will join in the memorial. It states that small contributions from a large number of engineers are preferable to a limited number of large contributions. Donations may be sent to the Engineering Foundation, at 29 West 39th St., New York City.



CONSTRUCTION PROGRESS, STONY GORGE DAM, CALIFORNIA

PRESENT state of construction of the Stony Gorge dam on the Bureau of Reclamation's Orlando project. The illustration shows the west side, with the spillway section in the foreground. The structure is 80 per cent complete. The contractor is the Ambursen Company.

Washington Notes

Flood-Control Bill Modified to Escape President's Veto—Federal Aid Bill Makes Little Progress

By Paul Wooton
Washington Correspondent

BY A VOTE of 254 to 91, the House of Representatives on April 24 passed the Jones flood-control bill, after having modified the measure to meet certain of the demands of the President. While several of the administration proposals were voted down, the consensus of opinion in Washington Wednesday night was that the President will sign the bill.

The measure provides that the federal government shall bear the entire cost of the project, including flowage rights and levee rights-of-way on the diversion channels. It was made incumbent on the government to control, confine and regulate these diversions. No change was made in the amount of the appropriation, which is fixed at $325,000,000. The House approved a one-civilian planning board, to which the Senate conferees are expected to agree.

An amendment by Representative Tilson, the majority leader, which would have had the effect of withholding compensation until actual flood damage has resulted—an amendment strongly urged by the President and approved by the Attorney General—failed by a vote of 67 to 119. The Sacramento River project was retained in the bill.

An effort to require the city of New Orleans to pay all damages arising from the construction of the Bonnet Carre spillway and to require southeastern Missouri and southwestern Illinois, jointly or severally, to do likewise in connection with the New Madrid floodway also failed. The vote was 72 to 143.

Determined efforts on the part of advocates of reservoirs to secure authority to use a portion of the appropriation for any reservoir that might meet the approval of those in charge were ineffectual, but an amendment was adopted providing that the reservoir survey would be made simultaneously with the work on the main river.

Boulder Dam Legislation

Boulder dam legislation is expected to occupy the Senate's attention during the greater part of the current week. All concerned admit that the vote in the Senate on the Swing-Johnson bill will be very close, with prospects slightly in favor of the passage of the measure. On the other hand, the temper of the House is against the bill. The failure of the St. Francis dam has given new support for the bill of Representative Dougias, of Arizona, which proposes a further study of the engineering phases of the question before Congress approves any project. This point of view has been strengthened decidedly by the recommendation of the special committee of American Engineering Council, of which Francis Lee Stuart is chairman.

As this is written, the rules committee of the House has given no indication as to when provision will be made for the consideration of the Muscle Shoals bill. It is apparent that the committee is very reluctant to give right-of-way to a measure such as has been reported out by the military affairs committee. It is thought to be practically certain that the President will veto it. In view of the situation, it is possible that some compromise may be worked out, but indications are that there will be no Muscle Shoals legislation at this session.

Highway Funds

No progress has been made, at this writing, in the effort to get the highway bill off the snag it has struck in the House. The bill passed by the Senate still is on the Speaker's table and the bill reported by the House committee is on the calendar, but five members of the committee have submitted a minority report recommending that federal aid be extended at the rate of $100,000,000 per year, rather than at the rate of $75,000,000 per year, which has had the effect of delaying consideration of the bill. In the Senate legislation is still pending providing for the construction of roads in towns, the building of certain interstate bridges, the concentration of money on interstate highways in Western states and the planting of trees along highways in states where the program is well advanced.

In the House it is feared that the legislation may be delayed dangerously in an effort to secure assurances that an amendment increasing the authorization of funds will not be offered. If the bill fails, state highway commissioners will be handicapped in letting their contracts for new construction.

Engineers and Geologist to Study Los Angeles Water System

The Board of Water and Power Commissioners of Los Angeles on April 20 appointed a board consisting of Louis C. Hill and R. E. McDonnell, engineers, and R. T. Hill, geologist, "to make a comprehensive survey of the water systems of the city to determine the necessity of reservoirs in the hills overhanging the populous sections of the city and for the further purpose of recommending ways and means for the substitution of other methods of meeting the city's water needs where this can be accomplished." The water board also passed a resolution agreeing to remove such reservoirs if the engineering board recommended a practicable method.

Independent action by the city council to reassure the public took the form of the appointment of another engineering board consisting of A. J. Wiley, F. C. Herrmann and C. H. Paul, who will investigate and report on the safety of existing dams, beginning with the Mulholland dam above Hollywood.

American Engineering Council Opposes Boulder Dam

Opposition to the Boulder dam project is declared by a special committee appointed by the American Engineering Council to examine the question from an engineering standpoint. The committee, of which Francis Lee Stuart of New York is chairman, says in its report, made public by the president of the council, Arthur W. Berresford:

"Your committee is unanimously agreed that the information set forth in the reports which have been submitted to us is not conclusive as to the engineering feasibility of the plan outlined in the Swing-Johnson bill and that before either the government or private capital would be justified on engineering or economic grounds in committing itself to the expenditure involved, the development of the river for the purpose in view should be further and most thoroughly investigated to determine how the result which it seeks to accomplish can be secured with a reasonable assurance of success."

Other members of the council's committee are Allen Hazen, Clemens Herschel, J. Waldo Smith and Lewis Buckley Stillwell.

Two-Day Spring Meeting at Utica of N. Y. Section, A.W.W.A.

In accordance with the recent vote of the New York Section of the American Water Works Association to have its meetings cover two days, the spring gathering will be held May 3 and 4. The meeting will be at Hotel Utica, Utica, N. Y., with the co-operation of the city and the Consolidated Water Company. After luncheon on May 3, there will be a session devoted chiefly to criticism of the association's Water Works Manual. Sections of the book on distribution, finance and management, and fire protection will be taken up respectively by E. K. Wilson, chief engineer, the Pitometer Company, New York City; Harry F. Huy, general manager, Western New York Water Company, Buffalo, N. Y., and George W. Booth, chief engineer, National Board of Fire Underwriters, New York City. At the close of this discussion there will be a superintendents' round table. In the evening entertainment will be provided by the Consolidated Water Company.

At the morning session, May 4, a paper on cross-connections will be presented by Charles W. Sherman, of Metcalf & Eddy, Boston, and one on protection of public water-works by Robert C. Wheeler, consulting engineer, Albany. John A. Korschen, commissioner of public works, Middletown, N. Y., will talk on the development of a municipal water supply. F. E. Beck, of J. G. White & Company, New York City, will relate the history of the water supply of Utica. After luncheon there will be an automobile trip to points of local interest by courtesy of the city of Utica.

County Road Officials to Discuss Standardization and Economy

Standardization of county road building and maintenance methods designed to save millions of dollars annually to the counties of the United States will be the objective of a national conference of county highway officials to be held in Washington, D. C., on May 12. The occasion is the first yearly meeting of the County Officials' Division of the American Road Builders' Association, and will be held following the general sessions of the American Road Builders' Association on the preceding day.

The meeting will be the first national business session of county highway officials ever held. It will be important because methods for economically spending some $600,000,000 annually for county road building and maintenance will be considered. More than $3,000,000,000 has been expended on the county road systems of the United States during the past five years. Before the organization of the County Officials' Division no effort had been made to standardize construction methods to produce the maximum of economy.

Thomas J. Wasser of Jersey City, N. J., president of the division, will preside.

G. W. Fuller Reports on Aeration of Wanaque Water

Utilization of the entire available head of the Wanaque reservoir, provision for aeration during a portion of the year, the aerated water to be lifted to the aqueduct, and the constant discharge of compensation water through separate fountains in the aerator are the main recommendations of a report by George W. Fuller, consulting engineer, New York City, to the mayor of Passaic, N. J., that city being one of the communities forming the North Jersey Water Supply District. The report is one of the sequelæ of the Wanaque aqueduct controversy of a year ago, a part of which centered in whether the water should be aerated.

Mr. Fuller proposes that the aqueduct and appurtenances should be built to take water direct from the reservoir when the water level is at or above 280 ft.; that pumps be installed to lift water to the aqueduct when the water level is below El. 280 or when the aerator is in use—which latter he assumes will need to be only during a small part of the year. He also assumes that the need for aeration may extend over longer periods as population on the tributary drainage area increases and that ultimately, though not for twenty years, filtration will be required.

Under the original plan, with the reservoir discharging directly to the aerator, the head on the aqueduct would be limited to 242 ft. Utilizing the higher head proposed, Mr. Fuller estimates, would save capital outlay for pumps for some of the municipalities, cut down pumping expenses in a considerable part of the district and, by giving more head, increase pressures and lessen the need for larger mains even at the lower levels. A portion of the savings would be offset by new construction requirements, but Mr. Fuller estimates that his plan would effect a total capitalized saving of "at least $1,000,000 and probably much more."

To put Mr. Fuller's recommendations into effect they would have to be approved by Passaic and the other communities in the district and by the North Jersey District Water Supply Commission. It is understood that the commission has already approved revised plans for the Wanaque aqueduct, prepared by Arthur H. Pratt and approved by Morris Sherrard, chief and consulting engineers to the commission, and that these plans, together with a request for authority to spend $5,000,000 more on the project, have been before the municipalities for some weeks.

Contract Awarded for Hudson River Bridge Tower Foundation

The first contract on the Manhattan end of the Hudson River Bridge was awarded by the Port of New York Authority on April 19. It was for the tower foundation and the anchorage in Fort Washington Park and was given to the Arthur McMullen Company, of New York, for $986,600. Thirty-two bids for the work were received (*Engineering News-Record*, March 8, 1928); and the bid of the McMullen company was the lowest.

When complete the tower foundation will consist of two granite-faced concrete piers, each 76x84½ ft. in plan. The construction of the anchorage as required at this time covers only that part necessary to permit erection of the steelwork. It consists of an irregularly shaped block of concrete of varying heights covering an area approximately 194x300 ft.

A.G.C. Board Meeting to Be Held in May

The regular spring meeting of the executive board, advisory board, presidents' council and secretaries' council of the Associated General Contractors of America will be held in Washington, D. C., May 7, 8 and 9. A continuation of the activities that were featured at the West Baden convention will mark the sessions of the May meeting. Some of these subjects include the following: False credits; fair practice in entering new territory; proper limitation of award on bidder's first construction contract with a public body; pre-qualification of bidders; co-operation with subcontractors; quantity survey, and building codes.

Prominent among the activities scheduled for the first session are the passing of a resolution of appreciation of services rendered by Gen. R. C. Marshall, Jr., during the eight years that he has been general manager of the association, and the installation of D. A. Garber as General Marshall's successor.

Preliminary Survey of Columbia and Lower Willamette Rivers

The Chief of Engineers has forwarded to the Secretary of War, for transmission to Congress, a report on the "preliminary examination and survey of Columbia and Lower Willamette rivers between Portland, Ore., and the sea," recommending modification of the existing project so as to provide for a channel 35 ft. deep and 500 ft. wide between Portland and the mouth, at an estimated cost of $1,366,000, with $685,000 annually for maintenance.

Technical Society Committees to Draft Building Code

Committees have been appointed recently in California to represent local sections of the American Society of Civil Engineers, the American Institute of Architects and Associations of General Contractors for co-operative endeavor in compiling a model building code for California municipalities. This action was taken when it developed that these societies were not in complete accord with the proposed model code recently brought out by the Pacific Coast Building Officials' Conference.

The California Development Association, a state-wide body, took sufficient interest in the matter to appropriate a minimum of $15,000 to defray expenses of the society committees, and the association will serve as a sort of clearing house for the committee activities. Expenditure of the funds, however, is entirely in the hands of the technical societies' executive committee. Careful consideration will be given to including in the code such provision for earthquake shocks as may be found to be desirable, and to this end the co-operation of the Seismological Society of America has been enlisted.

The three groups, comprising engineers, architects and contractors, have each named two representatives, one from the northern and one from the southern part of the state, to form a general committee of six, the chairmen of each society's committee to constitute the executive committee. A total of about 40 men, all told, will be contributing gratis their time and effort to this work. Representing the American Society of Civil Engineers two sections in the north and two in the south are included. The other societies have one section each in the northern and southern parts of the state.

At a preliminary meeting held in Santa Barbara on March 30 all six committee chairmen were present and a general program of procedure was outlined. To serve as editors of the new code it was decided to retain two men, one an engineer and the other an architect. The men selected for these positions are H. D. Dewell, consulting engineer, San Francisco, and Edwin Bergstrom, architect, Los Angeles. A paid assistant for each will be provided, together with all necessary funds for traveling expenses, clerical help, etc.

Engineering Societies

Calendar

Annual Meetings

NATIONAL FIRE PROTECTION ASSOCIATION, Boston, Mass.; Annual Meeting, Atlantic City, May 7-10.

AMERICAN ASSOCIATION OF ENGINEERS, Chicago; Annual Convention, El Paso, Tex., June 4, 5 and 6.

AMERICAN WATER WORKS ASSOCIATION, New York; Annual Convention, San Francisco, week of June 11.

SOCIETY FOR THE PROMOTION OF ENGINEERING EDUCATION; Annual Meeting, University of North Carolina, June 25.

AMERICAN INSTITUTE OF QUANTITY SURVEYORS, Chicago; Annual Convention, New York, June 25-27.

THE NEW YORK SECTION of the American Water Works Association will hold its next meeting at the Utica Hotel, Utica, on May 3 and 4. E. D. Case, 50 Church St., New York City, is secretary.

THE SOUTHERN CALIFORNIA CHAPTER, Associated General Contractors, has elected as its president Ford J. Twaits, of the Scofield Engineering Construction Company. E. A. Irish was chosen vice-president and George Hess secretary-treasurer.

THE BALTIMORE SECTION, American Society of Civil Engineers, met at the Baltimore Engineers Club on the evening of April 13, and listened to an address on "The Engineer and His Allies" by Willard Chevalier, general manager, *Engineering News-Record*. This was followed by an informal discussion of the suggestions offered concerning the training of engineers to deal with present-day economic conditions.

NEW YORK SOCIETY OF THE PANAMA CANAL—The fourth annual banquet of the society is to be held Saturday evening, May 5, at the Hotel Astor, New York City. Judge Frank Feuille, vice-president, will be presiding officer, with Jay J. Morrow as toastmaster. Other speakers will be Gerard Swope, Swagar Sherley and William Darling. The meeting will take up the matter of erecting a suitable memorial to the late Gen. George W. Goethals. A reception is also planned from 3 to 6 p.m. at the Roosevelt House, 28 East Twentieth St., to institute the memorial exhibition to General Goethals which is to be on display for several weeks.

THE CENTRAL STATES SECTION of the American Water Works Association will hold its 1928 convention in Huntington, W. Va., on Aug. 23-24.

THE WISCONSIN SECTION of the American Water Works Association will hold its annual meeting on Oct. 15 at Menasha, Wis. L. A. Smith, Madison, Wis., is secretary.

Personal Notes

GEORGE HAMLIN, chief engineer of maintenance of the Connecticut Highway Department, has been elected president of the Connecticut Civil Engineers Association.

Major BEVERLY C. DUNN, Corps of Engineers, has been relieved from duty in the office of the Chief of Engineers and assigned to duty in the office of the Assistant Secretary of War, Washington, D. C.

Lieut. Col. WILLIAM C. WEEKS, Corps of Engineers, U. S. Army, has been transferred from Omaha, Neb., to be U. S. district engineer at Chicago, succeeding Col. Edward H. Schulz.

GUSTAVE H. RADEBAUGH, manager of the Urbana and Champaign Sanitary District, Urbana, Ill., has been appointed director of the State Department of Conservation.

I. I. HINTON, of San Antonio, has been appointed head of the city sanitary department of Breckenridge, Tex. Mr. Hinton served in the marine corps as a sanitary engineer during the World War and has since served as sanitary engineer at Nacogdoches and in the Mississippi flood area between Memphis and New Orleans.

Col. WILLIAM KELLY, Corps of Engineers, U. S. Army, has resigned his commission, and the resignation has been accepted by the War Department. Colonel Kelly, who is vice-president of the Buffalo, Niagara & Eastern Power Corporation, has been on leave of absence from the army for the past three years. He was chief engineer of the Federal Power Commission from its formation until 1925, when he resigned to become director of engineering for the National Electric Light Association.

Obituary

JEREMIAH F. CAREY, office engineer under City Engineer A. K. Vickery, Denver, Colo., died on April 9 from asthma and heart trouble. He was about 60. Mr. Carey was a native of New Haven, Conn.

J. G. HARVEY, of Chicago, an engineer under contract by the Colombian government to supervise the construction of a steel bridge across the Magdelena River at Girardot, was drowned in Fuquene Lagune, Colombia, on April 9 after saving the lives of several women and children when their boat capsized. Mr. Harvey saved all but himself.

JOHN MOLITOR, city architect of Philadelphia and designer of virtually all the Sesquicentennial buildings, died on April 19 after a long illness caused by overwork in his office and his labors on the exposition buildings. He was 55 years old. Mr. Molitor was appointed city architect in 1924. Before that he had been chief of the division of housing and sanitation, to which position he was appointed in 1914, when it was created.

CLARENCE V. MILHOLLAND, 43 years old, a civil engineer in the valuation department of the Baltimore & Ohio Railroad, Baltimore, was killed on Sunday night, April 22, when he was struck by an automobile. A passer-by found him unconscious and had him removed to a hospital, where he was pronounced dead from a fracture of the skull. Mr. Milholland was a brother of Francis X. Milholland, assistant to the senior vice-president of the Baltimore & Ohio.

CHARLES WELLFORD LEAVITT, landscape engineer, designer of many public parks, towns, country clubs, racetracks and large private estates, died on April 22 at his home in Hartsdale, N. Y., from pneumonia. Mr. Leavitt was 57 years old. He began his professional work in civil engineering as an assistant engineer of the East Jersey Water Company. In 1890 he served as assistant engineer and later as chief engineer of the Caldwell Railway, now a part of the Erie system. At the time of his death Mr. Leavitt was a consulting landscape architect for the Board of Water Supply of New York City and for the North Jersey District Water Supply Commission.

HARRY CHARLES SANFORD, long prominent in tunnel and subway construction at New York, died on April 22, 1928, at his home in Edgewater, N. J., from pneumonia. Mr. Sanford was born in 1869 at Mantua, Ohio. He was graduated from Valparaiso University in 1889 as a civil engineer. During the course of his engineering career Mr. Sanford had been associated with a number of notable engineering projects in the fields of railroad and subway construction, water supply and tunnels. Among these were the Baltimore Belt Line tunnel, the Jerome Park reservoir of the New York City water supply, sections of the New York City subways, the Steinway tunnel under the East River, the Hudson & Manhattan tubes connecting New York and New Jersey, the Catskill aqueduct, the electrification of the Virginian Railway and the reconstruction of the New York & Ottawa Railway bridge across the St. Lawrence River. He was co-inventor of the Coughlin-Sanford swing-rail frog. Since 1925 Mr. Sanford had been associated with the Rosoff Subway Construction Company, working on sections of the new Eighth Ave. subway in New York City. He was a director of the Degnon Construction Company and of the Degnon Terminal Railway Company, vice-president of the Rosoff Subway Construction Company and vice-president, treasurer and director of the Universal Speed Control Company. He was also an associate member of the American Society of Civil Engineers.

Construction Equipment and Materials

A Section Devoted to What the Manufacturer Is Doing for the Engineer and Contractor

Americans Invited to Exhibit at Mexican Road Show

An invitation has been received by Charles M. Upham, director, American Road Builders Association, from the Mexican government inviting the Association to take part in the Road Building Demonstration and Exposition which will be held either the last week in September or the first week in October, 1928. Details as to cost of exhibit space, etc., are not available at the present time, although it is probable that special freight rates will be secured, and machinery can be shipped in bond so that no import duties need be paid on any machinery that is not sold. If enough manufacturers of highway equipment are interested in this exhibit, it is stated, steps will be taken to secure additional data.

Business Notes

PENNSYLVANIA-DIXIE CEMENT CORPORATION, New York City, has announced the purchase of the Pyramid Portland Cement Company, Des Moines, Iowa, with an annual output of 1,250,000 bbl. This additional output will bring the total for the Pennsylvania-Dixie corporation to 12,250,000 bbl. yearly. A plan for consolidation of the Pennslyvania-Dixie Cement Corporation and the North American Cement Corporation has been submitted to the stockholders for approval. Such a consolidation would bring under one management a combined annual capacity of 16,750,000 bbl.

THE TEXAS COMPANY, 17 Battery Place, New York, announces that it will loan to any responsible organization motion pictures showing the most modern methods of constructing sheet asphalt and asphalt macadam pavements.

EDGAR S. ROSS recently resigned from the Mellon Institute of Industrial Research, Pittsburgh, after serving for five years as senior fellow, roofing fellowship. He is now manager of research and development, Headley Good Roads Company, Philadelphia, on work incident to practical applications of emulsified asphalts.

THE AMERICAN FLURESIT COMPANY, Cincinnati, Ohio announces the following new distributors: Cincinnati Quarries Company, Cincinnati, Ohio; A. M. Stagg Fluresit Company, Jersey City, N. J.; Connecticut Fluresit Company, New Haven, Conn.; Fluresit Company of New England, Boston, Mass.; D. S. McKinlay, Syracuse, N. Y.; and D. E. Curtis, Lima, Ohio.

BENJAMIN BEAN, formerly structural steel designer for the Iowa Steel & Iron Company, Cedar Rapids, Iowa, is now designer in the engineering department of the Iowa Manufacturing Company, Cedar Rapids. The latter company manufactures material-handling equipment for highway building materials.

MATT BRODIE, manager for Asia of the Sullivan Machinery Company, Chicago, with headquarters at Tokyo, Japan, died recently from pneumonia contracted following an emergency operation for appendicitis. Mr. Brodie entered the employ of the Sullivan company after his graduation from the Massachusetts Institute of Technology in 1902. He had been successively manager of the Salt Lake City branch, manager of the Sydney, Australia, branch, in charge of the company's business in Russia and for the past 10 years in general charge of the company's business in all countries of Asia.

New Developments

Selling Grade-Marked and Squared Lumber in Packages

Innovations in both the manufacture and the distribution of lumber, carrying grade-marking one step further than heretofore, are being inaugurated by Weyerhaeuser Forest Products, St. Paul, Minn., with mills in northern Minnesota, Idaho and the Pacific Northwest. Lumber will be distributed in grade-marked packages and will be squared in the mill so as to eliminate much costly work in the field. At present the items to which the plan is being applied include bevel and Colonial siding, drop siding, rustic siding, softwood flooring, ceiling, end matched west coast hemlock flooring and a wide variety of moldings.

After the regular milling, these items are taken to a plant where newly invented re-butting saws are located. These saws are claimed to produce an absolutely smooth, four-square butt. Each end of a group of pieces is re-trimmed separately, being held against the guides by rubber-faced pneumatically operated clamps, while the saw is operated by electricity. Besides eliminating one of the most costly wastes in building, the four-square butt is calculated to eliminate many abuses of sound lumber construction principles.

Following re-inspection, the lumber is brought together in from three to twelve thicknesses, according to the kind, and the "container" is adjusted over it. This consists of a pair of heavy sulphate fiber caps that fit over the ends of the package and are fastened to the reverse sides of the top and bottom wrap-boards. These caps operate on a combination of the principles of the sleeve and the hinge, to accommodate the flexure of the package as it goes through various movements in shipping, warehousing and trucking. Each end-cap bears a bright-colored label on which is printed in plain English the kind, the species and the grade of the package's contents, together with the name of the particular producing mill. It is stated that lumber has heretofore been grade-marked only in code that has been understandable by only the lumbermen or others who have made a study of the subject.

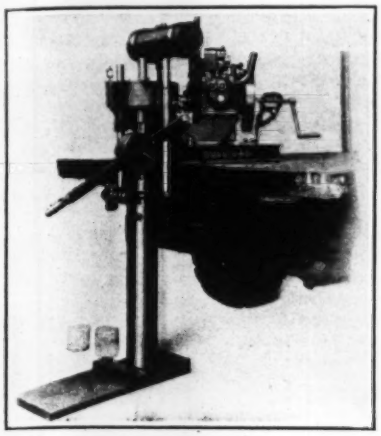
Drill for Concrete Pavement Core Sampling

To permit inspection of structure, materials and thickness of concrete pavements, the Sullivan Machinery Company, Chicago, has developed a new gasoline-operated core drill. The total assembly weighs 772 lb. and is intended for mounting on the rear end of a motor truck. The bit drills a 6⅛-in. hole, removing a 6-in. core at the rate of 1 ft. in fifteen to twenty minutes. With the core barrel usually furnished, the machine will drill to a depth of 18 in.

The machine consists of a drive rod rotated through bronze worm gears by a direct-connected gasoline engine. The drive rod is kept in alignment by substantial side rods connected to a cross-head at the bottom of the drive rod and running through cylindrical sleeves, which are cast in one piece with the heavy bedplate or frame supporting the drill and engine. The drill is fed downward automatically, although the speed is controlled by a weighted hand lever.

In starting a hole, a guide made of plank is laid on the pavement. The operator stands on the board and the core barrel passes through a hole just large enough to admit it. The bottom of the barrel is thus held firmly for

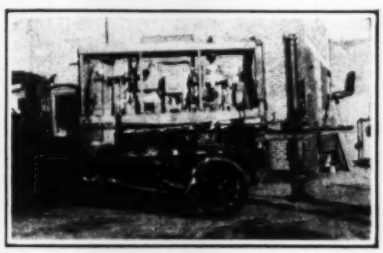
starting the hole. A small handful of shot and a pint of water are poured into the core barrel, which is open at the top. From time to time more shot and more water are added. The shot



finds its way into two vertical slots cut in the bottom of the core barrel, and from these roll under the face of the barrel. The speed of rotation recommended is about 100 r.p.m.

A New Truck-Mounted Pneumatic Pavement Breaker

A pneumatic pavement breaker of large size mounted on a turntable carried by a motor truck is a new machine being manufactured and sold



by the Rapid Pavement Breaker Corporation, 1718¼ Garfield Place, Los Angeles, Calif. A rotating carriage supports the compressor and the hammer so that the loads are balanced at all times regardless of the position of the machine. The machine is suitable

for crushing concrete and asphalt pavements, curbings and the floors of old concrete bridges. Considerable saving in time, man power and expense is claimed over hand-operated pavement breakers. The machine is equipped with an 8x6-in. 220-cu.ft.-per-minute Gardner-Denver compressor. It has been in operation in California for about two years.

The machine has a horizontal swing of 10 ft., and the cylinder head is mounted upon a trunnion so that it is allowed to swing within an arc of 2½ ft. forward and backward. The force of the machine's blow is controlled by a semi-automatic valve and can be varied from 10 lb. to 15,000 lb. The metal frame, which just clears the pavement, serves as a guide for cutting. Different types of cutters have been devised for different pavements. For concrete, the cutting shoe has a considerable number of points of contact, so that the concrete can be crushed into small enough pieces for use as sub-base. The breaker and compressor weigh approximately 3½ tons.

Railroad Spikes Pulled by New Air Machine

An air-operated spike puller has recently been developed by the Ingersoll-Rand Company, New York City. Operated by one man, it is claimed it will pull from eight to ten spikes per minute. The machine is light enough to be moved by hand from tie to tie.

The company states that because of low air consumption (about 3 cu.ft. of air per spike) this spike puller can be

run from its tie-tamper compressor without affecting the operation of other pneumatic appliances.

In operation the throttle is turned to admit air on top of a plunger. As the plunger moves down, it operates a link arrangement which opens the jaws on the lower end. With the jaws open, the machine is placed over the spike and the foot rest is placed on the lower flange of the rail. By rotating the throttle air is admitted beneath the plunger. As it rises the jaws close under the head of the spike. Upward movement of the jaws takes place only after they are firmly clamped to the spike.

Manufacturers and Trade Associations

Calendar

Annual Meetings

NATIONAL LUMBER MANUFACTURERS ASSOCIATION, Washington, D. C.; annual meeting, Chicago, Ill., April 30 to May 2.

AMERICAN WATERWORKS MANUFACTURERS ASSOCIATION, New York City; annual meeting, San Francisco, Calif., June 11-14.

New Publications

Speed Reducers — ALBAUGH-DOVER MANUFACTURING COMPANY, Chicago, Ill., has published a new bulletin on its speed reducers, illustrating and describing the units and including dimensional drawings and specifications for the units in ratios of 4, 5 and 6 to 100.

Air Compressors — SCHRAMM, INC., West Chester, Pa., has published a new 1928 catalog covering its complete line of air compressors and pneumatic tools. Besides illustrations and descriptions of the detail parts, many operating views are included, showing the various types of mountings and their use on work of diverse requirements.

Paint and Waterproofing — TOCH BROTHERS, New York City, have made available complete sets of their architects' and engineers' loose-leaf specifications arranged in a folder for convenient filing. The specifications cover most of the company's technical paints and waterproofing compounds, and of the latter both the membrane and integral types.

Overhead Crane Operation—WHITING CORPORATION, Harvey, Ill., has published a new booklet entitled "Operation and Maintenance of Whiting Electric Cranes." The booklet gives information on the erection and wiring of the cranes and then devotes a number of pages to correct methods of operation. All necessary spare parts are listed and many of them are illustrated.

Valves — WELLMAN-SEAVER-MORGAN COMPANY, Cleveland, Ohio, has published Bulletin No. 89 devoted to its Supertest gate and globe valves. Six of the 19 pages are devoted to illustrations and descriptions, while the remainder are blueprint sheets of valve dimensions. All valves are designed for the highest pressures and temperatures used in modern practice.

Business Side of Construction

Facts and Events That Affect Cost and Volume

Foreign Projects of Interest to American Firms

Among the items of highway news just issued by the Bureau of Foreign and Domestic Commerce are several that may be of interest to American manufacturers and contractors. The president of Chile has been authorized by law to contract for a loan for the city of Vina del Mar, for elimination of grade crossings, paving streets, and building 16 kilometers of road. A highway to connect Madrid and Valencia has been surveyed. It is expected that the building of the Madrid-Irun Highway will start this year. The construction of an international highway to connect Montevideo with Rio de Janeiro was proposed in the arrangement for the settlement of the balance of the Uruguayan debt to Brazil, and it is reported that the proposition was accepted. The state of Minas Geraes, Brazil, has allotted $782,748 for road building. The Road Commission of the Bureau of Highways of Kiangsi Province, China, has prepared a plan whereby seven main highways shall radiate from Nanchang, the capital, and a network of branch roads will connect all parts of the province with the capital.

The highway improvement program in Czechoslovakia adopted in 1927 contemplating an expenditure of $30,000,000 during the next ten years is important not only because of its influence on automotive development but also because of the interest being taken in American roadbuilding machinery, according to advance information from the Department of Commerce. Several American manufacturers, it is reported, are negotiating with local agencies, while a newly formed company has taken over representation of fourteen American manufacturers grouped under the auspices of an American export house. Albert Baldwin is the American attaché at Prague.

A proposed Italian highway from Venice to Trieste, 132 to 135 kilometers in length, is reported by Vice-Consul John E. Holler, Venice. The cost per kilometer would be approximately 1,120,000 lire, or 59,000.

This Week's Contracts

Heavy construction contracts, reported by *Engineering News-Record* in the week ended April 26, with some comparisons, total as follows:

(In Thousands of Dollars, 000 Omitted)

Week Ended	Public	Private	Total
Apr. 26 1928	**$35,957**	**$36,409**	**$72,366**
Apr. 19, 1928	38,838	42,613	81,451
Apr. 28, 1927	24,218	34,353	58,571
Jan. 1 to date:			
1928	$372,895	$721,598	$1,094,493
1927	308,203	569,681	877,884

E. N.-R. Index Numbers

Cost		Volume	
Apr. 1, 1928	**206.40**	**March, 1928**	**318**
Mar. 1, 1928	204.65	February, 1928	225
Apr. 1, 1927	209.00	March, 1927	226
Average, 1927	206.24	Average, 1927	263
Average, 1926	208.03	Average, 1926	228
1913	100.00	1913	100

Industrial Contracts Maintain Lead Over 1927

Value of industrial building contracts of $40,000 and more let in the first quarter of 1927 exceeds that of a year ago by 55 per cent. The number of jobs in each quarter compare strikingly—317 in 1928 and 316 in 1927. These average $252,094 and $175,962, respectively.

Public utilities continue to lead the fourteen industrial groups, and here also there is agreement in number—29 this year and 27 last. Gains are registered by nine groups. The textile and clothing increase is due mostly to a $10,000,000 rayon project at Richmond for E. I. du Pont de Nemours & Company.

The lead was gained in February, and March lettings further boosted it. It should be noted, too, that all except the Far West participated in the increase, which is heaviest in the Middle Atlantic states. The figures are given in the accompanying table.

INDUSTRIAL BUILDING CONTRACTS IN FIRST QUARTERS

	1928	1927
Public Utilities:		
Value	$26,861,000	$17,850,000
Number	29	27
Chemical Process:		
Value	9,843,000	3,086,000
Number	38	31
Food Products:		
Value	5,057,000	4,761,000
Number	24	36
Automotive:		
Value	6,938,000	3,694,000
Number	53	48
Machinery and Equipment:		
Value	2,973,000	9,697,000
Number	28	59
Textiles and Clothing:		
Value	11,331,000	1,518,000
Number	19	20
Printing and Binding:		
Value	187,000	1,350,000
Number	4	8
Steel Mills:		
Value	1,434,000	310,000
Number	6	2
Cement and Quarry Products:		
Value	1,029,000	1,600,000
Number	11	10
Foundries:		
Value	419,000	973,000
Number	6	5
Refrigeration and Cold Storage:		
Value	4,913,000	2,248,000
Number	20	22
Lumber and Woodworking:		
Value	745,000	670,000
Number	6	5
Warehouses:		
Value	3,273,000	2,550,000
Number	15	16
Unclassified:		
Value	4,911,000	5,297,000
Number	58	27
Total:		
Value	79,914,000	55,604,000
Number	317	316

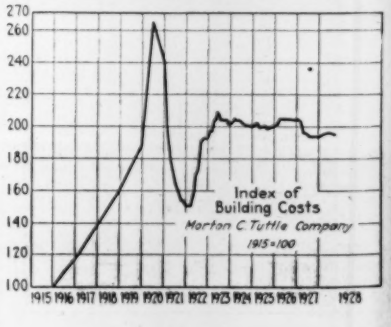
Business Notes

Among the week's real estate bond offerings are three large issues for hotels: the Mayflower, in Washington, $7,000,000; the Governor Clinton, $5,000,000, and an apartment hotel, $4,450,000, both in New York. In the April 12 issue the $50,000,000 offering of the American Gas & Electric Company was noted. The construction program of that company, just announced, will cost $46,000,000 in 1928. The principal items are the completion of the $17,000,000 extension of the Philo power station of the Ohio River Company and the $12,000,000 Deepwater generating station in southern New Jersey. Some recent offerings are listed below:

		Interest Rate	Yield
Public Utility:			
Calif. Water Service	$6,254,000	5	4.80
Municipal:			
Alabama	5,000,000	4–4½	3.85–4
Bridgeport, Conn.	450,000	4	3.85
Fort Worth, Tex.	1,550,000	4½	4.05
Houston, Tex.	120,000	6	4.50
Schenectady Co., N.Y.	300,000	3.90	3.75
Tennessee	3,000,000	3½–4	3.85–3.98

Steel ingot production for the first quarter of 1928 was about 3 per cent above the 1927 quarterly total. The average daily output for all companies was 160,823 gross tons during the first three months of 1928 and 157,626 tons during the same period in 1927. Per cent operation was 87.39 this year to date, compared with 85.66 per cent a year ago. Following are the March output figures in tons for the United States and Great Britain:

	United States	Great Britain
March, 1928	4,407,520	793,000
February, 1928	4,045,304	764,400
March, 1927	4,535,720	949,600

One year ago British steel mills were actively engaged in meeting orders that accumulated during the coal strike. Last month's output was larger than the monthly average of any other year since the war. French output in February totaled 738,000 tons, compared with 628,000 in February, 1927. This is greater than for the average month of any year since the war. Steel production in January in ten countries showed only one under the average monthly rate of 1927.

The Morton C. Tuttle index, plotted below, is obtained by re-estimating monthly the cost units of a concrete building constructed in southern New England in 1914. The number, therefore, represents the variation between the original cost and what it would cost to replace the same building today.